DEFINITE PROJECT REPORT/ENVIRONMENTAL ASSESSMENT

AROOSTOOK, FORT FAIRFIELD, MAINE

MAIN REPORT / SUPPORTING DOCUMENTATION

LOCAL FLOOD PROTECTION

DRAFT REVIEW

AUGUST 1987

US Army Corps of Engineers
New England Division

AROOSTOOK RIVER

FLOOD DAMAGE REDUCTION PROJECT

FT. FAIRFIELD, MAINE

TECHNICAL APPENDICIES

FOR

DEFINITE PROJECT REPORT

FOR

WATER RESOURCES DEVELOPMENT

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS 02254-9194

AUGUST 1987

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HYDROLOGIC ANALYSIS

AROOSTOOK RIVER FLOOD CONTROL
FORT FAIRFIELD, MAINE

HYDROLOGIC ANALYSIS
FOR
DETAILED PROJECT REPORT

by

HYDROLOGIC ENGINEERING SECTION
WATER CONTROL BRANCH
ENGINEERING DIVISION

NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

AROOSTOOK RIVER FLOOD CONTROL
FORT FAIRFIELD, MAINE

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FORT FAIRFIELD, MAINE

HYDROLOGIC ANALYSIS

1. PURPOSE

This report presents hydrologic information and analysis pertinent to the planning and design of flood control improvements along the Aroostook River within the town of Fort Fairfield, Maine. Included are sections on watershed description, climatology, flood frequencies, analysis of floods, and improvements for flood control.

2. WATERSHED DESCRIPTION

The Aroostook River is a tributary of the Saint John River in northern Maine and western New Brunswick, Canada. Its watershed is situated between those of the Penobscot and Allagash Rivers to the west; the Fish River to the north; the Saint John to the east, and the Meduxnekeag River to the south. The Aroostook River originates at the junction of the Munsungan and Millinocket streams in the northwest corner of Penobscot County, Maine and flows in a general northeasterly direction for about 100 miles through Aroostook County before crossing the international boundary below Fort Fairfield. After crossing the international boundary, the Aroostook River flows an additional five miles in an easterly direction through New Brunswick, Canada to its confluence with the Saint John River at Aroostook Junction, Canada. Of the total drainage area of 2,418 square miles, approximately 2,300 square miles lie upstream of Fort Fairfield, which is essentially the entire portion of the basin within Maine. A hydroelectric project, Tinker Dam, with a drainage area of 2,370 square miles and a head of 85 feet, is located 2 miles downstream of the international boundary. The Aroostook is a flat, heavily forested, hydrologically sluggish watershed having a total fall of about 450 feet in its 107 mile watercourse to the Saint John River. However, 85 feet or about 19 percent of the total fall is at Tinker Dam with the remaining 365 feet occurring as a rather uniform slope over the 105 mile river course above Tinker Dam. A map of the Aroostook River watershed is shown on Plate 1.

3. CLIMATOLOGY

a. General. The climate of the Aroostook River basin is cold and semi-humid with an average temperature of about 40° Fahrenheit and yearly precipitation of approximately 37 inches. Due to its northerly location, the area has escaped the brunt of coastal hurricanes with their accompanying intense rainfall. The area does experience periods of moderate rain and/or snowfall as a result of low pressure systems moving up the east coast and from frontal systems moving from west to east across the country.

b. Temperature. Average monthly temperatures in the basin vary considerably throughout the year. Summers are cool with temperatures averaging 60 to 65° Fahrenheit with only occasional rises into the nineties. Winters are long and cold with temperatures averaging 10 to 20° Fahrenheit. The mean, maximum and minimum monthly temperatures at two stations in the Aroostook River watershed, as published by the National Oceanic and Atmospheric Administration (NOAA), are summarized in Table 1.

TABLE 1

MONTHLY TEMPERATURE
(Degree Fahrenheit)

	Caribou, Maine 46 Year Record Elevation 624 FEET/NGVD			Presque Isle, Maine 70 Year Record Elevation 599 FEET/NGVD		
Month	Mean	Max	Min	Mean	Max	Min
January	10.0	51	-32	11.5	54	-41
February	13.1	47	-41	13.4	51	-37
March	23.9	58	-20	24.8	65	-30
April	36.9	80	2	38.0	85	-2
May	50.3	91	19	51.2	94	19
June	59.9	96	30	60.7	95	25
July	65.2	95	40	66.1	97	37
August	62.7	95	34	63.8	99	31
September	53.9	91	23	55.1	90	21
October	43.2	79	14	44.4	84	8
November	31.0	68	-2	31.6	69	-15
December	15.3	58	-24	16.7	58	-35
Annual	38.8	41.8	36.2	39.8	50.7	29.6

c. Precipitation. The average annual precipitation over the Aroostook River watershed is about 37 inches and is distributed rather uniformly throughout the year with slightly greater amounts during the summer months. Periods of moderate rainfall are usually not more than 1 to 2 days in duration and storm rainfall amounts generally do not exceed 1 to 2 inches. Monthly and annual precipitation for two locations within the Aroostook River watershed are shown in Table 2.

d. Snowfall. Practically all winter precipitation occurs as snow with the total fall averaging about 100 inches per year. Snow survey data for the watershed is limited but based on information gathered in adjacent

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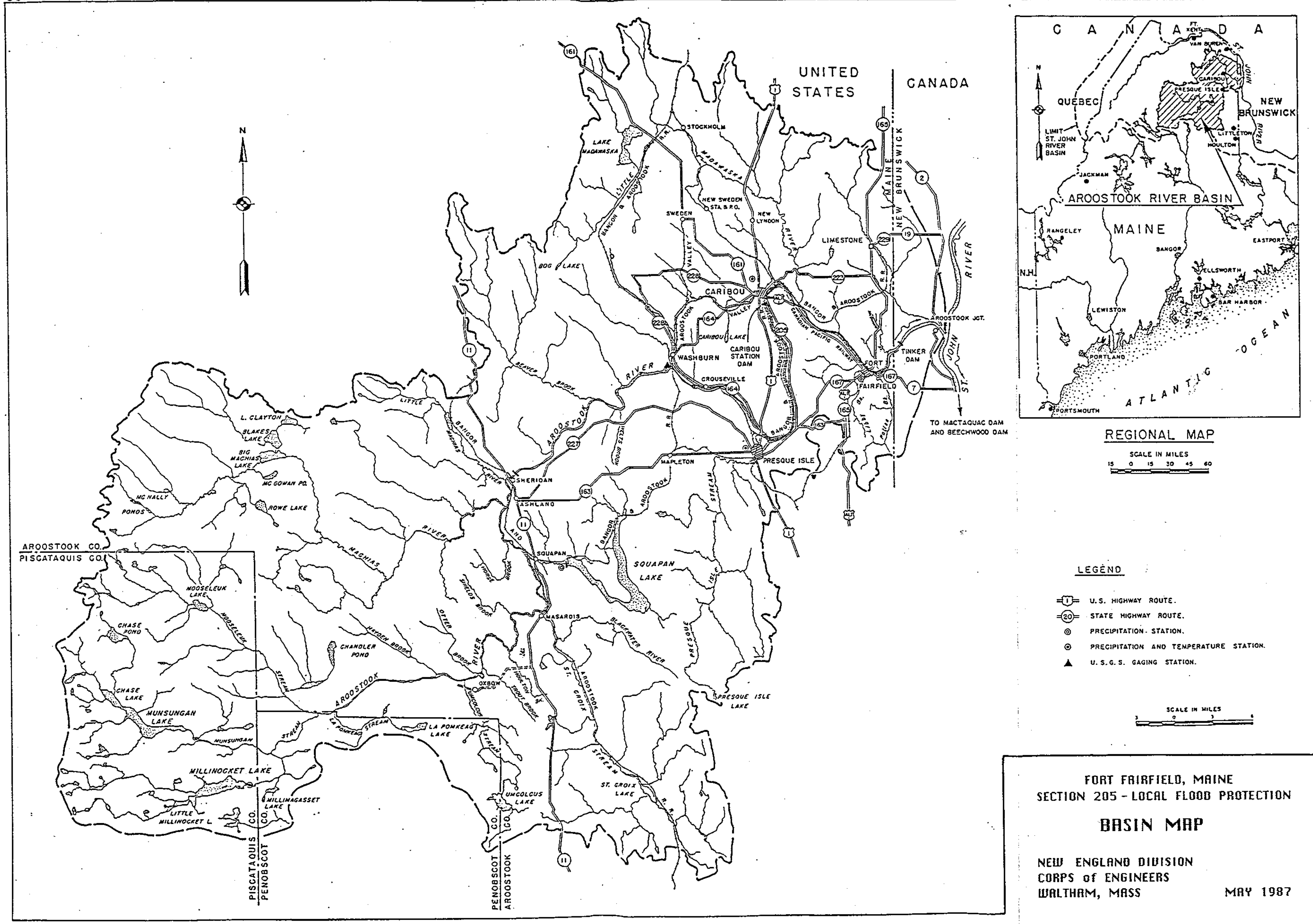

FORT FAIRFIELD, MAINE
SECTION 205 - LOCAL FLOOD PROTECTION

BASIN MAP

NEW ENGLAND DIVISION
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WALTHAM, MASS

MAY 1987

PLATE 1

basins the snowpack generally reaches a maximum in April. Average water equivalent of the spring snowpack is about 8 inches with maximums as high as 15 inches. Table 3 lists the mean, monthly and annual snowfall for two locations in the watershed.

TABLE 2

MONTHLY PRECIPITATION
(Inches)

	Caribou, Maine 46 Year Record Elevation 624 FEET/NGVD			Fort Fairfield, Maine 48 Year Record Elevation 300 FEET/NGVD		
Month	Mean	Max	Min	Mean	Max	Min
January	2.23	5.10	0.12	2.64	5.52	0.38
February	2.11	4.13	0.26	2.40	5.38	0.19
March	2.50	5.13	0.66	2.60	5.82	0.52
April	2.59	5.26	0.54	2.80	5.15	0.85
May	30.3	6.27	0.47	3.13	6.87	0.94
June	3.47	7.11	0.88	3.69	7.62	1.41
July	4.06	6.83	1.75	4.26	7.44	1.62
August	3.94	12.09	0.93	3.64	7.90	1.23
September	3.27	8.14	0.86	3.78	7.75	0.79
October	3.17	6.35	0.63	3.52	7.65	0.98
November	3.40	8.15	0.45	3.47	7.36	0.39
December	3.00	7.97	0.74	3.25	7.40	0.82
Annual	36.77	51.10	27.92	39.18	55.27	27.95

TABLE 3

MONTHLY SNOWFALL
(Inches)

	Caribou, Maine 39 Year Record Elevation 624 FEET/NGVD			Fort Fairfield, Maine 19 Year Record Elevation 300 FEET/NGVD		
Month	Mean	Max	Min	Mean	Max	Min
January	23.3	41.4	2.2	22.8	40.1	3.6
February	22.2	41.0	4.4	17.4	37.5	4.0
March	19.7	47.1	6.1	19.7	45.5	4.0
April	8.3	24.4	T	5.9	17.0	1.0
May	0.8	10.9	0	0.3	4.0	0
June	0	T	0	0	T	0
July	0	0	0	0	0	0
August	0	0	0	0	0	0
September	0	T	0	0	0	0
October	2.1	12.1	0	1.4	10.0	0
November	12.1	34.9	1.5	7.5	24.0	1.0
December	23.8	59.9	6.5	25.1	41.0	3.6
Annual	112.3	181.1	59.6	94.9	141.6	58.9

4. STREAMFLOW

a. General. Average streamflow in the Aroostook basin is about 1.6 cfs per square mile of drainage area, which is equivalent to about 22 inches of runoff per year or about 60 percent of average annual precipitation. Streamflow, however, is quite variable seasonally. Much of the winter precipitation occurs as snow, which does not run off but accumulates as deep snowpack. As a result, over 50 percent of the annual runoff occurs during the April-May spring snowmelt period. Maximum streamflow rates on the Aroostook River have been as high as 26 cfs per square mile of drainage area and lows frequently approach 0.1 cfs per square mile, generally occurring in late summer or the dead of winter.

b. Streamflow Records. There are no long term streamflow records for the Aroostook River in Fort Fairfield, Maine. There is, however, a long term USGS gaging station on the main stem Aroostook River located upstream of Fort Fairfield in the town of Washburn, Maine. This gage, with a drainage area of 1,654 square miles or 68 percent of the total Aroostook River watershed, has continuously recorded discharges since 1931. The discharge record at this station was used extensively in analyzing the hydrologic characteristics of the Aroostook River in Fort

Fairfield. Table 4 lists average monthly runoff as recorded at the Washburn gage. Average monthly runoff varies from about 6 inches in May to 0.6 inch in August and February. Extremes in monthly runoff have ranged from a high of over 14 inches in May to a low of 0.06 inch in February. In addition, annual peak flows at the gage are listed in Table 5.

TABLE 4

MONTHLY RUNOFF
AROOSTOOK RIVER AT WASHBURN, MAINE
D.A = 1,654 SQUARE MILES
(Continuous Recording Period 1931-1983)

Month	Mean		Maximum		Minimum	
	cfs	Inches	cfs	Inches	cfs	Inches
January	1010	0.70	2595	1.81	167	0.12
February	1000	0.63	3684	2.32	101	0.06
March	1432	1.00	10440	7.28	324	0.23
April	7693	5.19	16990	11.46	1468	0.99
May	8518	5.94	20350	14.18	3229	2.25
June	2556	1.72	5931	4.00	658	0.44
July	1402	0.98	5882	4.10	261	0.18
August	1042	0.73	5728	3.99	152	0.11
September	1169	0.79	5235	3.53	144	0.10
October	1752	1.22	8098	5.64	265	0.18
November	2501	1.69	9767	6.59	218	0.15
December	1848	1.29	7975	5.56	175	0.12
Annual	2666	21.88	4145	34.02	1409	11.56

5. FLOOD HISTORY

a. General. Floods along the Aroostook River have occurred to varying degrees over the years resulting from intense rainfall, snowmelt or ice jams, or from combinations of the three. The main flood season on the Aroostook River occurs in the spring when the chance of significant rainfall, and/or high temperatures, during the spring snowmelt period, pose an annual flood threat. As indicated by the listing of annual peak flows in Table 5, about 90 percent of the annual high flows occur during the spring months of March through May. In addition, ice jams are a major flood hazard every spring as well as being a major threat to bridge crossings and other structures.

The largest recorded discharge at Washburn was 43,400 cfs and occurred in April 1983. This flow was slightly greater in magnitude than the previous discharge of record of 43,100 cfs that occurred in April 1973. Available records indicated significant ice jam flood events occurred in the Fort Fairfield area in April 1976, March 1936, April 1940,

TABLE 5

ANNUAL PEAK DISCHARGES
AROOSTOOK RIVER AT WASHBURN, MAINE
(Drainage Area = 1,654 Square Miles)

Date	Discharge (cfs)	Date	Discharge (cfs)
13 Apr 1931	13,500	16 May 1960	25,000
13 Apr 1932	20,900	16 May 1961	37,000
4 May 1933	24,000	17 Jul 1962	19,200
21 Apr 1934	36,200	3 May 1963	23,000
		10 Nov 1963	29,200
1 May 1935	23,300		
22 Mar 1936	37,800	2 May 1965	7,670
30 Apr 1937	15,500	26 Apr 1966	14,500
22 Apr 1938	17,500	5 May 1967	18,400
11 May 1939	30,100	16 Apr 1968	17,800
		11 May 1969	27,600
14 Apr 1940	30,900		
22 Apr 1941	27,100	26 Apr 1970	25,600
28 Apr 1942	26,000	5 May 1971	28,000
13 May 1943	24,400	17 May 1972	24,200
11 Nov 1943	16,300	30 Apr 1973	43,100
		1 May 1974	42,800
3 Apr 1945	21,000		
28 Apr 1946	22,900	8 May 1975	20,200
8 May 1947	31,800	4 Apr 1976	32,200
20 May 1948	14,200	25 Apr 1977	27,200
18 Apr 1949	14,000	30 Apr 1978	19,200
		30 Apr 1979	37,700
23 Apr 1950	22,100		
30 Nov 1950	23,000	16 Apr 1980	13,400
30 Apr 1952	18,700	18 Aug 1981	17,200
3 Apr 1953	32,600	28 Apr 1982	31,500
29 Jun 1954	32,400	19 Apr 1983	43,400
		2 Jun 1984	25,500
6 May 1955	20,200		
16 May 1956	12,500		
24 Apr 1957	13,500		
25 Apr 1958	35,400		
27 Apr 1959	13,600		

and December 1973. Following are discussions of five of the more notable floods that have occurred within Fort Fairfield over the past 20 years. Flows at Fort Fairfield are generally proportioned to those at Washburn by a ratio of drainage area.

b. April 1973. Between 22 and 30 April 1973, 3.06 inches of precipitation combined with daytime temperatures in the sixties produced high discharges on the Aroostook River. The peak discharge recorded on the 30th at the USGS gage in Washburn was 43,100 cfs, the second largest flow in the 53 year period of record, and the Maine Public Service Company reported a flow of 60,800 cfs at Tinker Dam. Ice flows on the river during this flood contributed to flood damages but peak flood levels were due to the abnormally high river flows unaffected by any jams. The estimated peak flow at Fort Fairfield was 58,100 cfs.

c. December 1973. On Thursday, 20 December, over 3 inches of snow and about 0.3 inches of rain fell with temperatures below freezing. On Friday, temperatures warmed to near 50^{o} Fahrenheit and an additional 0.85 inches of rain fell resulting in a one-half mile long ice jam in the Fort Fairfield area with flooding to about 7 feet above normal river levels. This flood was principally an ice jam event with a maximum discharge, at the Washburn gage, of only about 14,000 cfs.

d. May 1974. During the period 28 April to 1 May, 1.35 inches of rain fell in the Aroostook basin and with daytime temperatures in the sixties, the third highest flow of record (42,800 cfs on 1 May) was experienced on the Aroostook River at Washburn, occurring only one year following the flood of April 1973. Though ice flows occurred during this event, the resulting flood was due mostly to the abnormally high river flows. The estimated peak flow at Fort Fairfield was 57,700 cfs.

e. April 1976. Probably the most devastating flood on the Aroostook River in the Fort Fairfield area occurred during the period 3-6 April 1976 and was the result of high flows with extensive ice jams comprised of ice chunks up to 43 inches in thickness. The peak discharge at the Washburn gage was 32,200 cfs with the stage surcharged about 3.7 feet by ice jams. At Fort Fairfield, the peak discharge was estimated to be 43,400 cfs with stage surcharged about 5 feet by a massive ice jam resulting in the record flood stage at Fort Fairfield of 365.6 feet NGVD. The April 1976 event was the result of about 1.6 inches of rainfall on 2 through 4 April in combination with daytime temperatures in the forties.

f. April 1983. The April 1983 flood was the result of about 1.6 inches of rainfall occurring between the 16th and 19th of April, preceded by a period of above normal temperatures plus snowmelt, thereby providing high antecedent runoff conditions. The resulting peak discharge recorded on the 19th at the USGS gage at Washburn was 43,400 cfs, the largest flow in the 53 year period of record. The estimated peak flow at Fort Fairfield was 58,500 cfs.

g. <u>Ice Jams</u>. Ice jams are practically an annual event in the Aroostook basin occurring during spring ice-out or at other times during the winter when freshets and temperatures are sufficient to cause river sheet ice breakup. Most frequently ice jams occur during rising riverflows and are broken up by time of peak discharge. The surcharge in river level caused by ice jams can be appreciable. Frequently, peak annual river levels are a result of ice jams occurring at times other than peak discharge. An analysis of peak annual stages and discharges at the Washburn USGS gage indicated that in 22 years out of 53, or 42% of the years, peak annual river levels were the result of ice jams. There obviously were many ice jam occurrences other than those producing peak stage for the year.

Peak annual stages, at the Washburn gage for the period 1931-1983, with those caused by ice jams noted, are listed in Table 6.

Ice jams on the Aroostook River at Fort Fairfield have added significantly to the flood problems of that community. The record flood stage at Fort Fairfield occurred in April 1976 as a result of an ice jam event during a high flow period. The resulting flood levels were about 3 feet higher than those produced by the non-ice jam flood of April 1973.

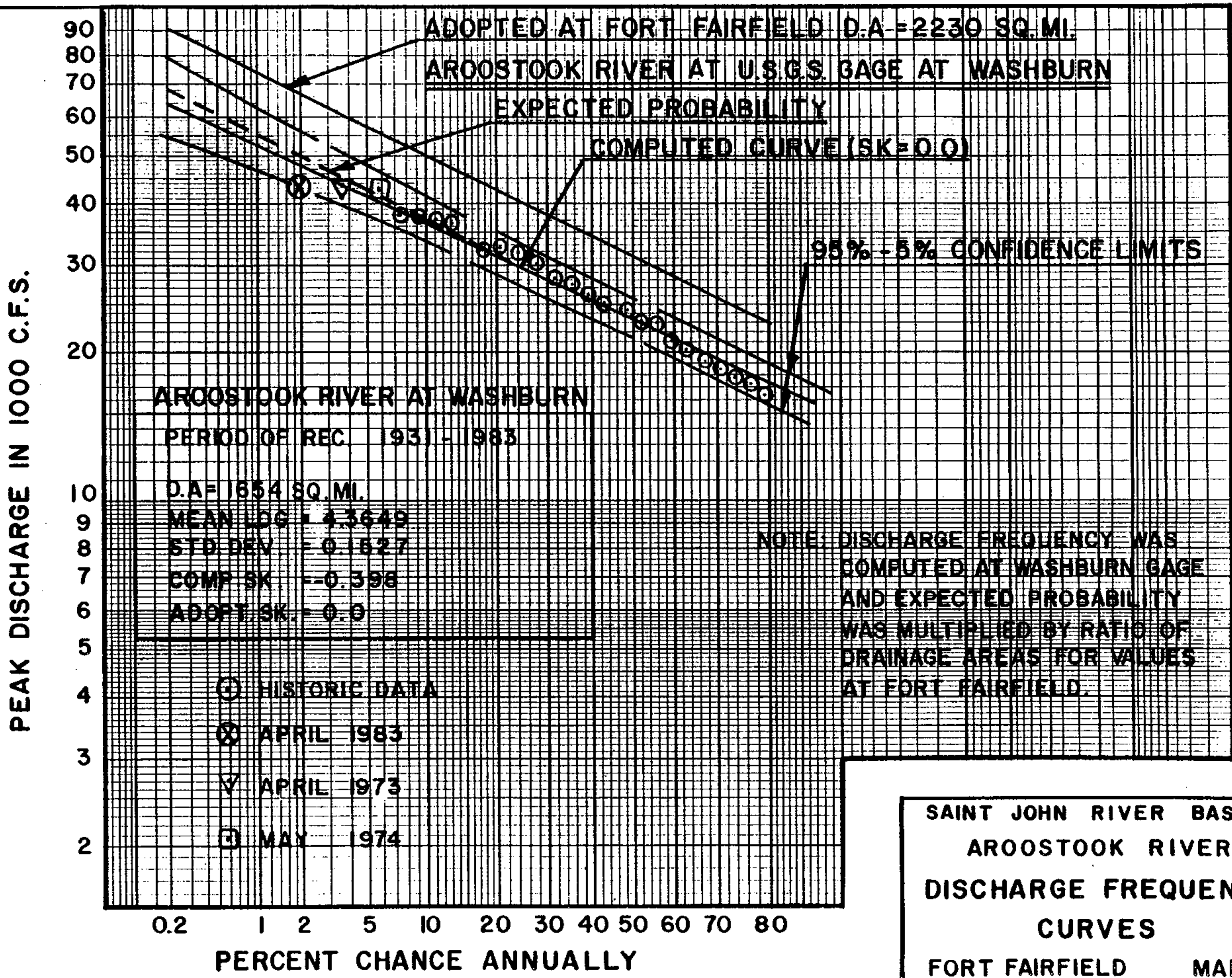
6. FLOOD FREQUENCIES

Peak discharge frequencies for the Aroostook River were developed by statistical analysis of long term peak flow records in the region. Discharge frequencies were developed for the Aroostook River, at the USGS gage in Washburn, Maine, by statistical analysis using a Log Pearson type III distribution in accordance with guidelines set forth in U.S. Water Resources Council Bulletin 17B, "Guidelines for Determining Floodflow Frequency," revised September 1982. The distribution of peak flows at the Washburn gage, with a drainage area of 1,654 square miles and a period of record of 53 years, had a mean log of 4.3649, a standard deviation of 0.1527, and a computed negative skew 0.40; however, a skew of 0.0 was adopted based on regional studies and broader data bases afforded by such analyses. Discharge frequencies at Fort Fairfield (D.A = 2,230 square miles) were considered proportional to those at Washburn by ratio of respective drainage areas. Other miscellaneous discharge data were also considered in arriving at the adopted frequency relation for the Aroostook River at Fort Fairfield. These data included 7 years of record (1904-1910) at a USGS gage at Fort Fairfield. The adopted discharge frequency curves with expected probability adjustment at Washburn and Fort Fairfield are shown on Plate 2.

TABLE 6

AROOSTOOK RIVER
PEAK ANNUAL STAGES
WASHBURN USGS GAGE
(1931-1984)

Date	Stage (ft)	Elevation (ft,NGVD)	Comment
13 Apr 1931	8.00	444.4	
13 Apr 1932	9.40	445.8	
4 May 1933	9.89	446.3	
21 Apr 1934	11.65	448.0	
20 Apr 1935	10.69	447.1	Ice Jam
22 Mar 1936	11.80	448.2	
30 Apr 1937	8.37	444.8	
21 Apr 1938	8.60	445.0	
11 May 1939	10.00	446.4	
15 Apr 1940	13.50	449.9	Ice Jam
17 Apr 1941	10.53	446.9	Ice Jam
28 Apr 1942	8.84	445.2	
13 May 1943	8.47	444.9	
11 Nov 1943	6.70	443.1	
3 Apr 1945	7.78	448.2	
30 Apr 1946	10.00	446.4	Ice Jam
8 May 1947	9.58	446.0	
20 May 1948	6.05	442.4	
9 Apr 1949	9.34	445.7	Ice Jam
23 Apr 1950	10.03	446.4	
6 Apr 1951	15.78	452.2	Ice Jam
20 Apr 1952	10.51	446.9	Ice Jam
30 Mar 1953	13.95	450.3	Ice Jam
29 Jun 1954	11.84	448.2	
22 Dec 1954	13.40	449.8	Ice Jam
18 Apr 1956	8.03	444.4	Ice Jam
24 Apr 1957	7.72	444.1	
21 Dec 1957	14.90	451.3	Ice Jam
11 Apr 1959	11.46	447.9	Ice Jam
16 May 1960	10.40	446.8	
16 May 1961	12.67	449.1	
17 Jul 1962	9.20	445.6	
3 May 1963	10.00	446.4	
10 Nov 1963	11.23	447.6	
2 Dec 1964	9.04	445.4	Ice Jam
26 Apr 1966	8.00	444.4	
5 May 1967	9.00	445.4	
13 Apr 1968	10.17	446.6	Ice Jam
11 May 1969	10.91	447.3	
26 Apr 1970	10.52	446.9	
24 Apr 1971	11.23	447.6	Ice Jam
17 May 1972	10.24	446.6	
30 Apr 1973	13.68	450.1	
24 Dec 1973	20.91	457.3	Ice Jam
22 Apr 1975	9.46	445.9	Ice Jam
3 Apr 1976	14.74	451.1	Ice Jam
3 Apr 1977	10.85	447.2	
22 Apr 1978	11.93	448.3	Ice Jam
30 Apr 1979	13.17	449.6	
11 Apr 1980	8.65	445.1	Ice Jam
6 Apr 1981	11.17	447.6	Ice Jam
22 Apr 1982	15.67	452.07	Ice Jam
19 Apr 1983	13.73	450.13	
17 Apr 1984	13.46	449.86	Ice Jam



SAINT JOHN RIVER BASIN

AROOSTOOK RIVER

DISCHARGE FREQUENCY CURVES

FORT FAIRFIELD MAINE

PLATE A-2

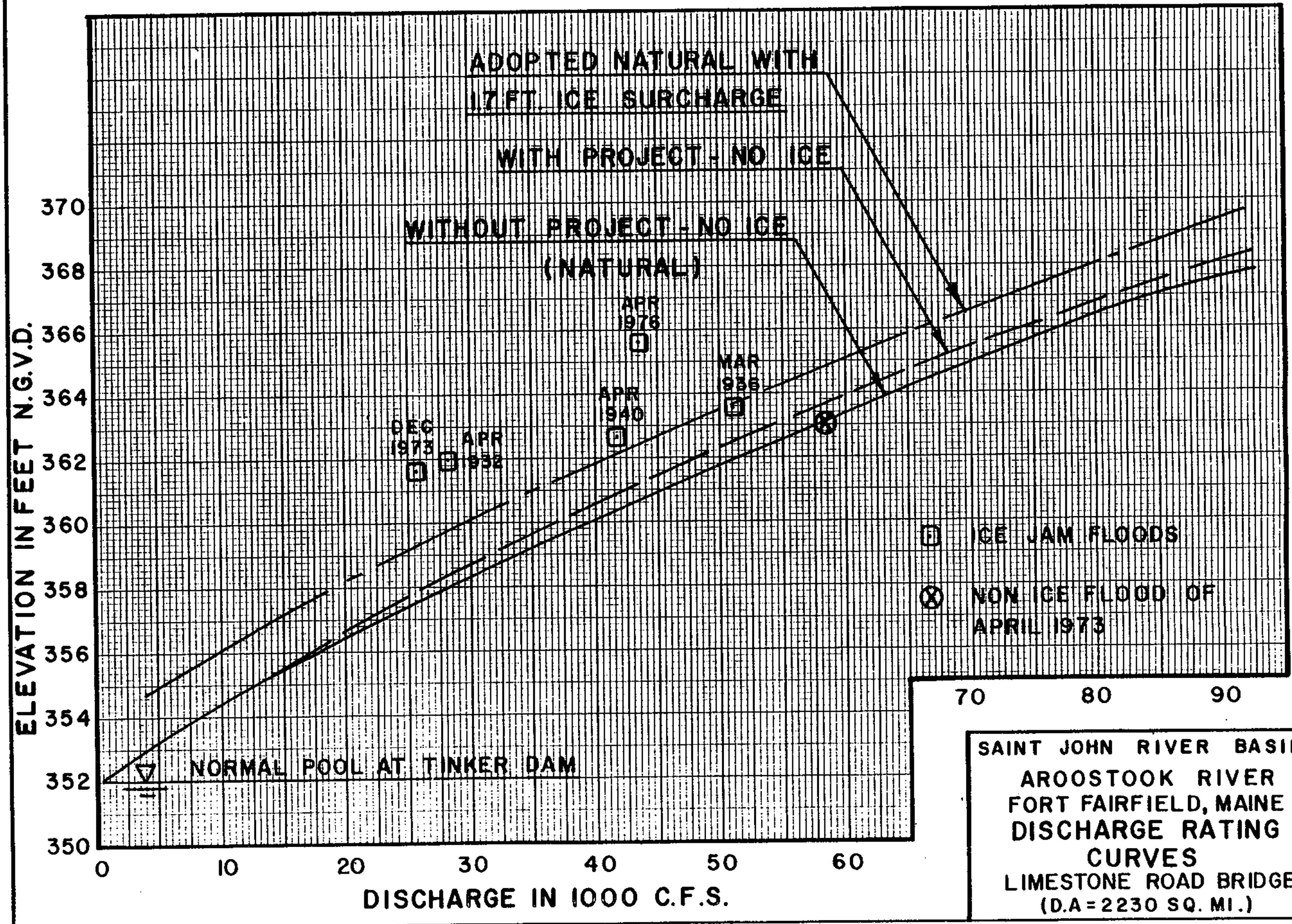
7. ANALYSIS OF FLOODS

a. General. The large drainage area of the Aroostook River at Fort Fairfield (2,230 square miles) is a hydrologically "sluggish" basin affecting peak discharges and duration of flooding at Fort Fairfield. Because of the large size of the watershed and its character, several days may pass before effects of heavy rains cause peak flows in the study reach. In the same manner, severe flood conditions may persist for as much as a week while reservoirs, lakes and large tracts of land in the southwestern Aroostook basin drain to normal levels.

b. Flood Profiles. Backwater flood profiles on the Aroostook River in Fort Fairfield were computed starting at the United States-Canadian border at river station 231+26 and proceeding upstream a distance of about 4 miles to river station 460+00 (about 1 mile above Limestone Road bridge). The starting water surface elevations for the Aroostook River were determined by developing a discharge rating curve at Tinker Dam, Canada located about 1 mile below the U.S.-Canada border. Information pertinent to the development of the rating curve was furnished by the Maine Public Service Company, Presque Isle, Maine. Tinker Dam is viewed more as a diversion than a high dam facility whereby the power potential of a natural falls is harnessed. The dam is equipped with a main spillway about 270 feet long with a 10 foot high bottom hinged gate. The invert of the gate is elevation 342 feet NGVD and normal maximum pool (gate raised) is elevation 352 feet NGVD. Flood stage discharge ratings were developed assuming the gate fully lowered and a main spillway weir coefficient of 3.6 with flow occurring over concrete non-overflow sections.

Backwater computations were made using cross section data as well as 5 foot contour mapping used in an earlier Aroostook River flood plain information report completed by NED in 1978. Computations were made using the Corps' HEC-2 computer program with Manning's "n" roughness coefficients of 0.03 for channel and 0.07 for overbank areas. Expansion and contraction coefficients were generally 0.3 and 0.1, respectively. The backwater model was calibrated by comparing developed discharge ratings at the Limestone Road bridge in Fort Fairfield with historic highwater marks of the April 1973 flood. A plan and profile of the Aroostook River in Fort Fairfield is shown on Plate 7.

c. Stage Discharge Relations. Stages of floods at Fort Fairfield are a function of not only the magnitude of flows but of the coincidences of ice jams. Normal or "non-ice" stage discharge relationships were first developed using the HEC-2 backwater model. A developed rating curve for the Aroostook River at the Limestone Road bridge in Fort Fairfield is shown on Plate 3. Also shown on Plate 3 is the adjustment to the "non-ice" curve to reflect the probable effect of ice. Adjustments consisted of increasing stages by 1.7 feet based on historic data of ice jams at Fort Fairfield as well as analyses performed at the Washburn gage as discussed in greater detail in paragraph 7d - Stage Frequency Relations.



SAINT JOHN RIVER BASIN
AROOSTOOK RIVER
FORT FAIRFIELD, MAINE
DISCHARGE RATING CURVES
LIMESTONE ROAD BRIDGE
(D.A = 2230 SQ. MI.)

PLATE A-3

d. Stage Frequency Relations. Flood stage frequency curves are conventionally determined directly, using developed discharge frequencies and a stage discharge rating for the river. However, because of the history of ice jams on the Aroostook River, stage frequencies were developed by analysis of both peak discharge frequencies and the frequency and magnitude of ice jam flood stages. Normal or "non-ice" stage frequency curves were first developed utilizing the developed peak discharge frequency curves and the "non-ice" stage discharge rating at the Limestone Road bridge in Fort Fairfield. At Fort Fairfield, historic data on ice jam stages is tabulated in Table 7. This historic data indicated that ice jams increased river levels an average of about 3.5 feet over non-ice levels. From inspection of the historic stages at the Washburn gage in Table 6, about 50 percent of the annual peaks were affected by ice. Therefore, the probable ice-affected stage frequency curve, at Fort Fairfield, was assumed mid-way (50 percent) between the non-ice stage frequency curve and the 100 percent ice surcharged curve or the non-ice stage frequency curve was adjusted upward 1.7 feet (50% x 3.5) to reflect the probable effect of ice. As a comparative check, coincident frequency procedures, as presented in Draft EC1110-2-249, dated 5 June 1985, were also performed and found to be in general conformance with the ice adjusted stage frequency curve. The adopted stage frequency curves are shown on Plate 4.

TABLE 7

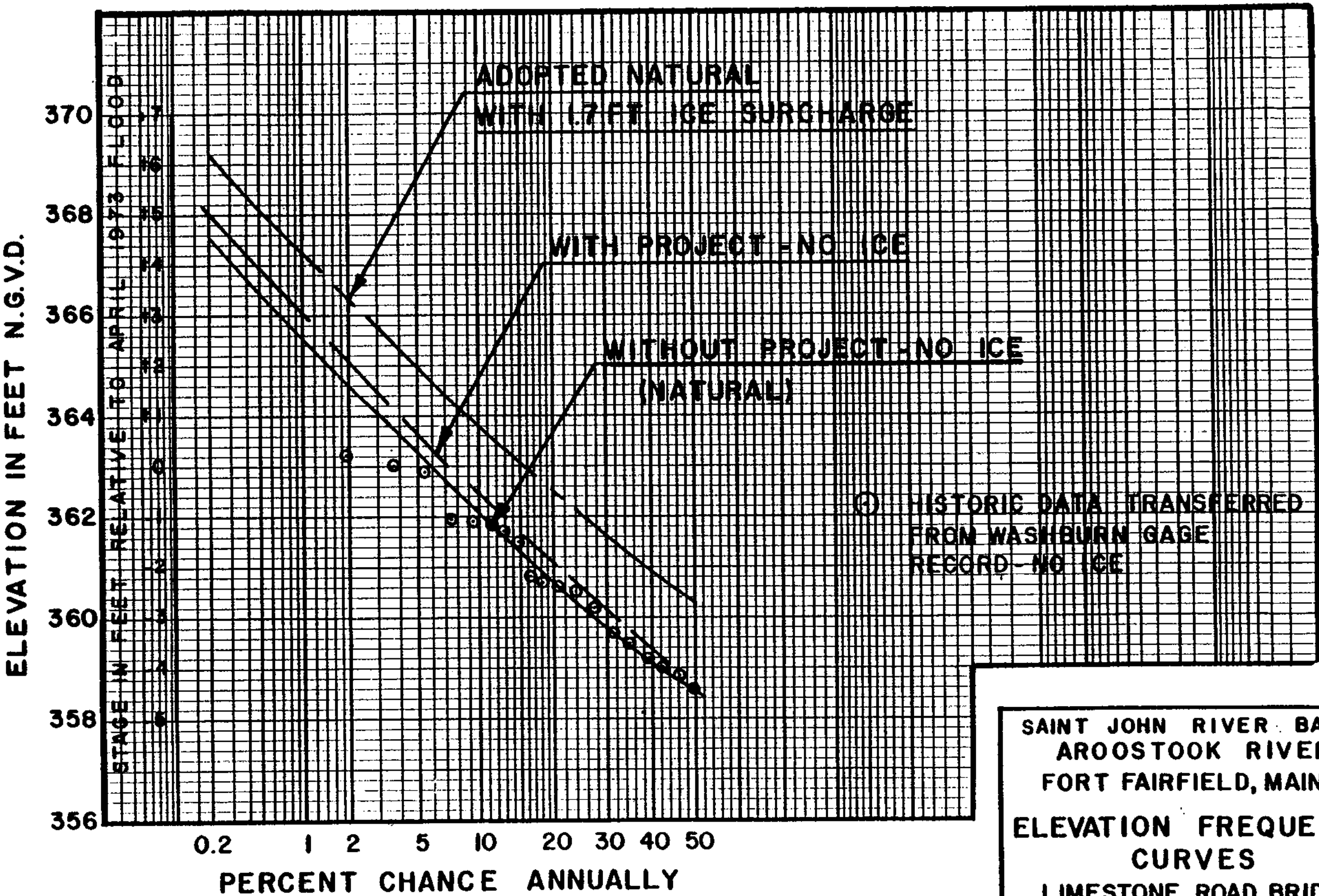
HISTORIC ICE JAMS
AROOSTOOK RIVER AT LIMESTONE ROAD BRIDGE
FORT FAIRFIELD, MAINE

Date	Historic "Ice" High Water (FT-NGVD)	Peak* Discharge (CFS)	"Non-Ice"** High Water (FT-NGVD)	Increase*** (FEET)
9 Apr 1932	362.0	28200	358.2	3.8
19 Mar 1936	363.5	51000	362.0	1.5
17 Apr 1940	362.7	41700	360.5	2.2
24 Dec 1973	361.7	25600	357.7	4.0
3 Apr 1976	365.6	43500	360.7	4.9

* Peak Discharge based on coincident peak flow at Washburn gage (DA = 1,654 sq. mi.) transferred to Fort Fairfield (DA = 2,230 sq. mi.) by drainage area ratio.

** Non-ice high water determined from peak discharge and developed "non-ice" discharge rating curve at Limestone Road bridge.

*** Increase in feet of "ice" high water over "non-ice" high water.



SAINT JOHN RIVER BASIN
AROOSTOOK RIVER
FORT FAIRFIELD, MAINE

ELEVATION FREQUENCY CURVES

LIMESTONE ROAD BRIDGE
(D.A = 2230 SQ. MI.)

PLATE A-4

8. STANDARD PROJECT FLOOD

a. General. Recommended flood control improvements for the Aroostook River at Fort Fairfield will not provide for Standard Project Flood (SPF) protection; however, an estimated SPF was developed as a "standard" against which the flood potential of the river could be judged, in comparison to the estimated frequency and magnitude of experienced floods. The SPF represents the flood discharge that may be expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the region excluding extremely rare combinations. The SPF for the Aroostook River at Fort Fairfield was developed by applying standard project storm rainfall to an adopted unit hydrograph generally in accordance with EM 1110-2-1411.

b. Standard Project Rainfall and Snowmelt. Standard project storm rainfall for the watershed above Fort Fairfield was determined from data developed during the Dickey-Lincoln School project studies. Since the Aroostook River is adjacent to and characteristically similar to the Dickey watershed (Saint John River), data developed during these studies were considered applicable for Fort Fairfield. In 1966, a report entitled: "Probable Maximum Precipitation for the Saint John River above Dickey Damsite and between Dickey and Lincoln School Damsites, Maine," was prepared by the Hydrometeorological Branch of the Office of Hydrology, U.S. Weather Bureau, Washington, DC. In this report, probable maximum precipitation (PMP) for six-hour periods and for drainage areas up to 5,150 square miles was presented for the subject basin. Probable maximum storm rainfalls were also developed for various seasons as a percentage of the all-season maximum. It was considered that approximately one-half of the PMP amounts would be appropriate for standard project storm (SPS) estimates for the basin above Fort Fairfield. A spring season (May) SPS rainfall of 4.3 inches in 24 hours was adopted for the watershed above Fort Fairfield. Assuming an infiltration rate of 0.2 inch per 6 hour period, a May SPS excess of 3.5 inches resulted.

In addition to the spring rainfall, snowmelt was considered in determining total SPF runoff. Runoff from snowmelt was determined by the following equation in accordance with EM 1110-2-1406:

$$M = 0.09 + (0.029 + 0.0084\ KW + 0.007\ R)\ (T-TF)$$

where

M = daily snowmelt in inches
K = exposure constant (1.0 for unforrested; 0.3 for forrested)
W = wind speed in MPH
R = daily rainfall in inches
T = air temperature in degrees Fahrenheit
TF = snowpack temperature in degrees Fahrenheit (usually $32^{\circ}F$)

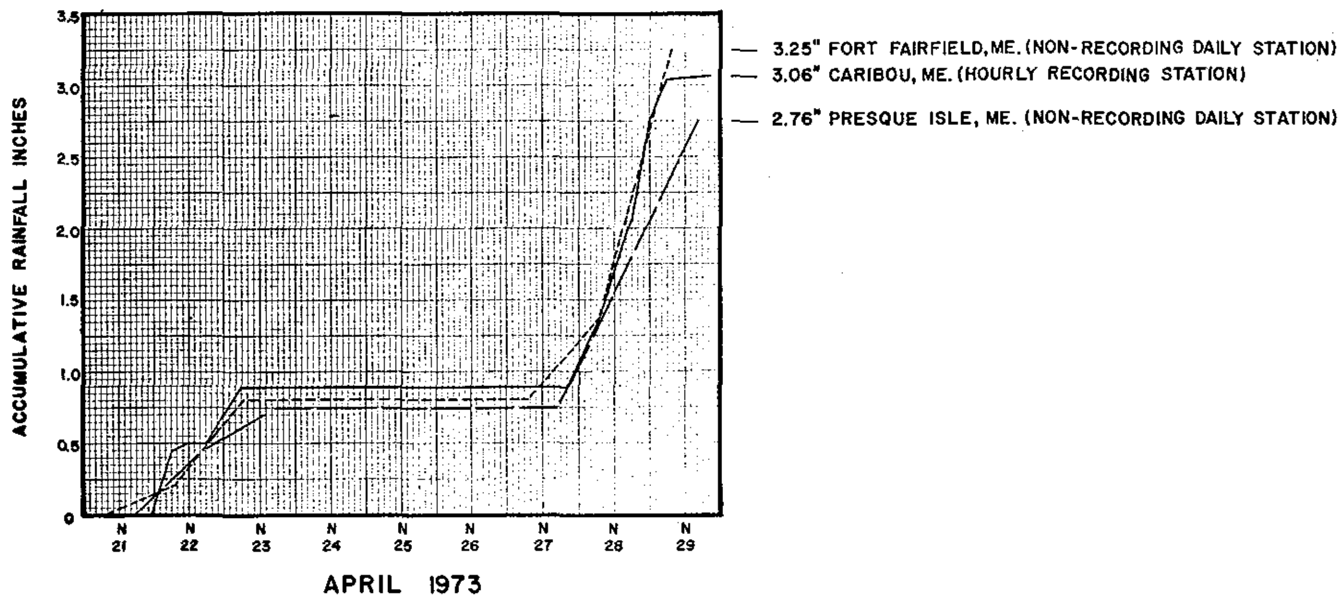
Applying the 24-hour SPS rainfall to the above equation, with a 10 MPH wind speed and 49°F air temperature, resulted in a 24-hour snowmelt of 1.52 inches. Therefore, the spring SPS excess of 3.5 inches coincident with a snowmelt of 1.52 inches resulted in a total SPF runoff of 5.02 inches. Six hour rainfall, losses, snowmelts, and excesses are listed in Table 8.

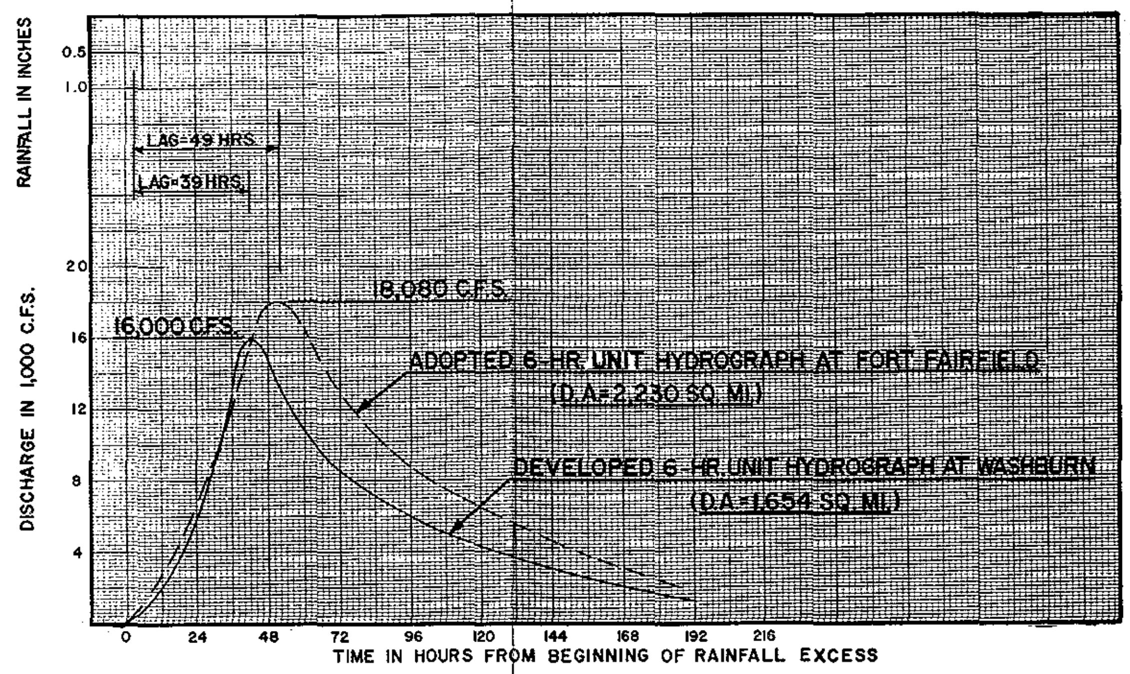
TABLE 8

SPRING SEASON STANDARD PROJECT FLOOD RUNOFF
AROOSTOOK RIVER BASIN
FORT FAIRFIELD, MAINE

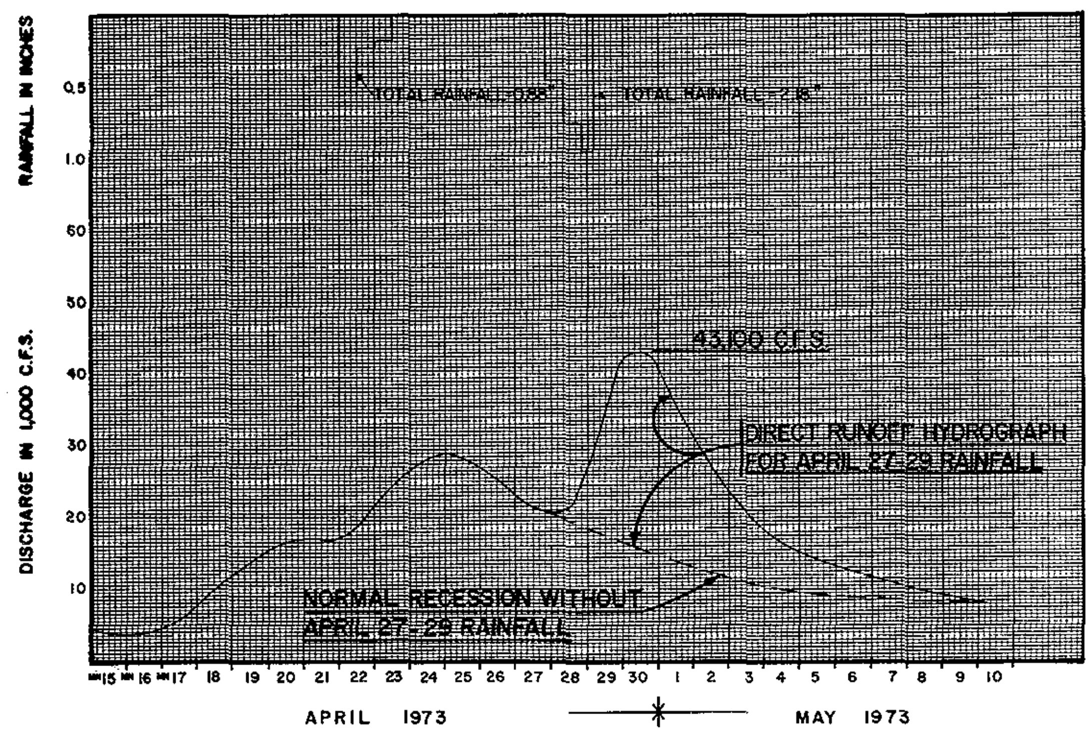
Time (hr)	Rainfall (in)	Loss (in)	Snowmelt (in)	Excess (in)
0-6	2.8	0.2	0.38	2.98
6-12	0.9	0.2	0.38	1.08
12-18	0.3	0.2	0.38	0.48
18-24	0.3	0.2	0.38	0.48
Total	4.3	0.8	1.52	5.02

c. Unit Hydrograph. A unit hydrograph for the Aroostook River at Fort Fairfield was developed by analysis of the April 1973 flood hydrograph at the USGS gaging station in Washburn. In April 1973, coincident with spring snowmelt, 0.87 inch of rain occurred on the 22nd and 23rd followed by 2.18 inches on the 27th thru 29th. The resulting flood hydrograph at Washburn had two distinct peaks on the 24th and 30th, with the latter being the second highest flow of record (see Plate 5). In developing the unit hydrograph, runoff from rainfall on the 22nd and 23rd was subtracted from the total hydrograph and the unit hydrograph was determined for the remaining hydrograph containing 2.4 inches of excess rainfall resulting from the storm on 27 thru 29 April. It is noted that the combination of the earlier storm together with substantial snowmelt during the first few weeks in April produced high antecedent conditions, resulting in a high percentage of rainfall runoff. The 2.4 inch runoff hydrograph had a peak discharge of about 27,000 cfs and was used to determine a 30-hour unit hydrograph. This 30-hour unit graph was then converted to a 6-hour unit graph by standard "s" curve procedures. To adjust the unit hydrograph for Fort Fairfield, Snyder's synthetic unit hydrograph coefficients were determined from the Washburn unit graph and prorated to Fort Fairfield. The resulting 6-hour unit graph at Fort Fairfield had a peak flow of 18,000 cfs and is shown in Plate 5. Pertinent unit hydrograph data are presented in Table 9.



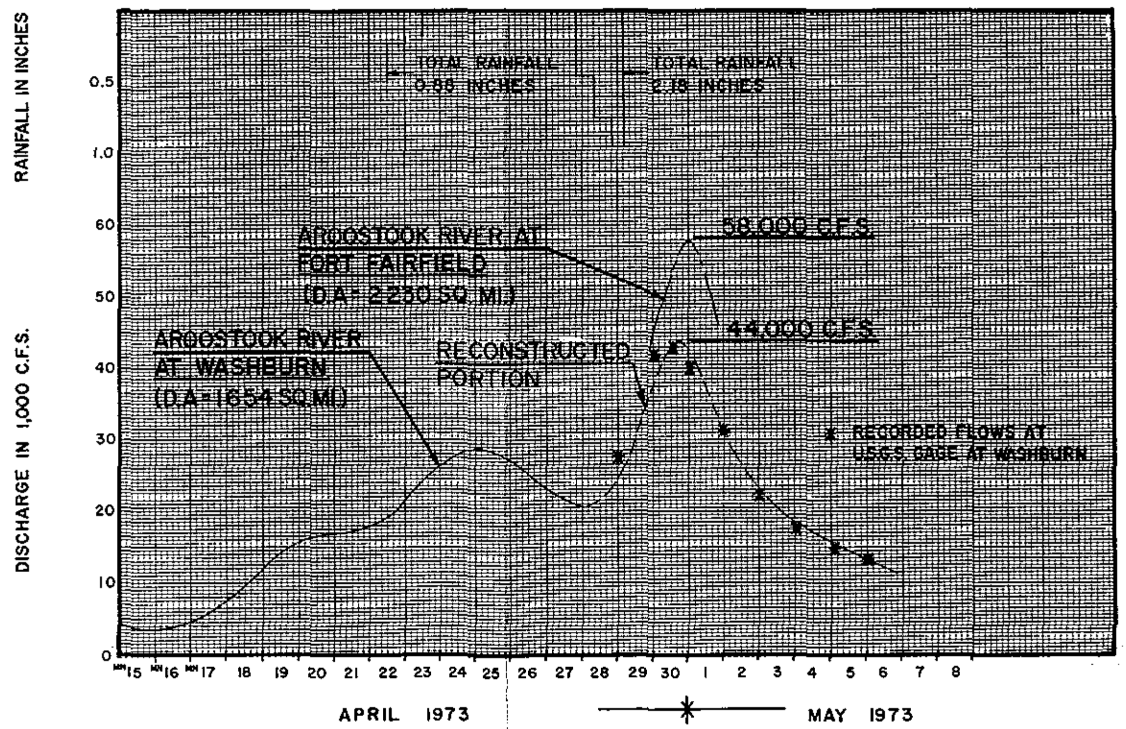


6-HOUR UNIT HYDROGRAPHS



U.S.G.S. GAGE WASHBURN, ME.

(D.A. = 1654 SQ. MI.)



RECONSTRUCTED FLOOD OF APRIL 1973

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

SAINT JOHN RIVER BASIN
AROOSTOOK RIVER

FORT FAIRFIELD, MAINE

UNIT HYDROGRAPH ANALYSIS

APRIL 1973 FLOOD

TABLE 9

AROOSTOOK RIVER
PERTINENT UNIT HYDROGRAPH DATA

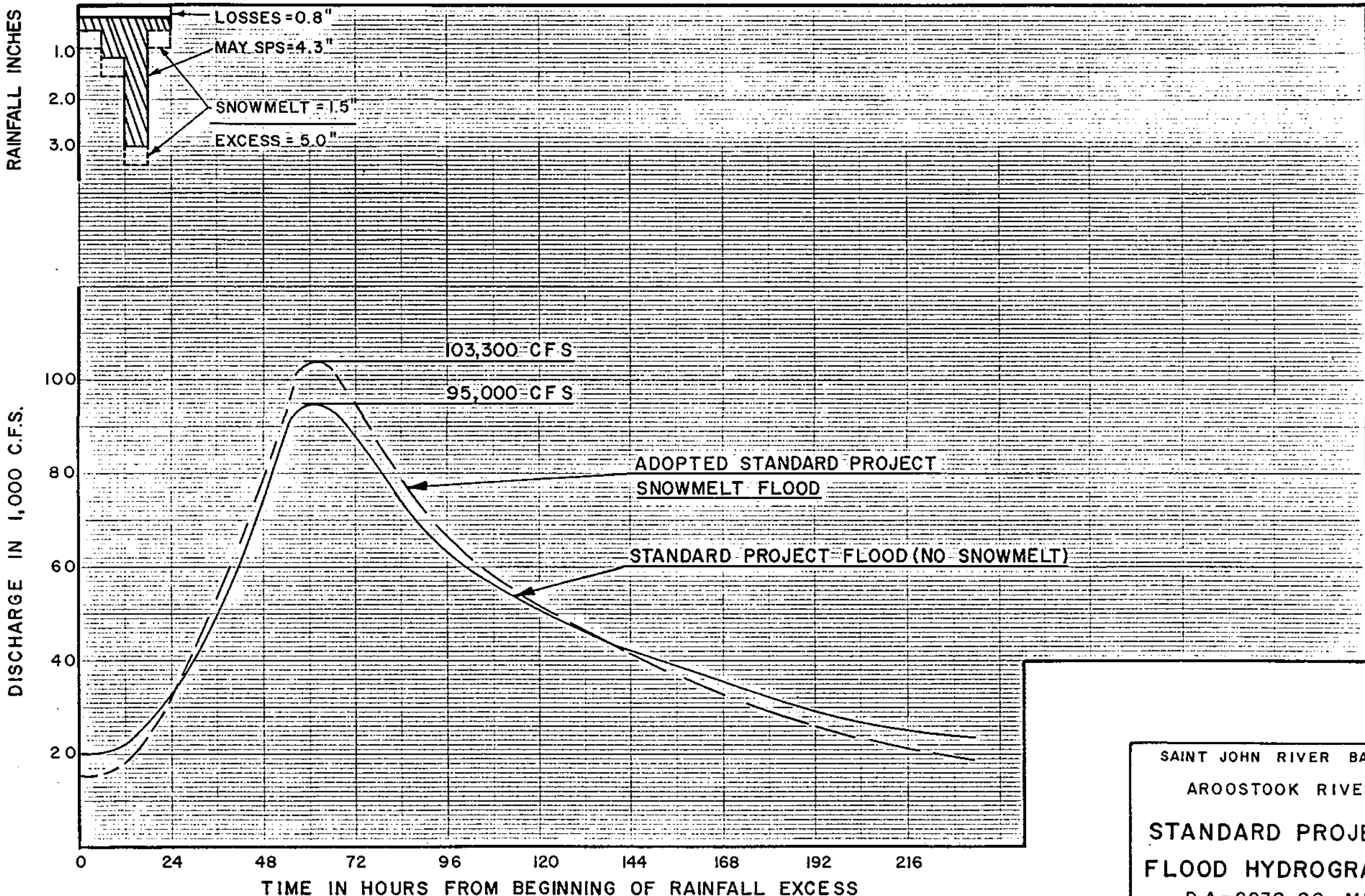
	Washburn	Fort Fairfield
Drainage Area	1,654 sq. mi.	2,230 sq. mi.
L	80 miles	112 miles
L_{ca}	36 miles	53 miles
T_r	6 hours	6 hours
T_p	39 hours	49 hours
C_T	3.5	3.5
C_p640	380	380
Q_p	16,000 cfs	18,000 cfs
q_p	9.7 cfs/sq. mi.	8.1 cfs/sq. mi.

d. Standard Project Flood. The spring season standard project storm rainfall plus snowmelt of 5.02 inches, applied to the adopted 6-hour unit hydrograph at Fort Fairfield, resulted in a spring season standard project flood discharge of 103,300 cfs as shown on Plate 6. The developed standard project flood discharge is about twice the magnitude of the April 1983 discharge of 58,000 cfs. The elevation of the standard project flood at Fort Fairfield would be about 370 feet NGVD or about 4 feet higher than the record April 1976 ice jam related flood or about 7 feet above the non-ice jam related flood of April 1983 flood. A plan and profile of the standard project flood, both with and without improvements, are shown on Plate 7.

9. FLOOD CONTROL IMPROVEMENTS

a. General. Following initial reconnaissance studies, feasible structural improvements for flood control consisted of earth dike flood protection along the rightbank of the Aroostook River within the town of Fort Fairfield. Protection would start about 2,900 feet downstream of the Limestone Road bridge and continue upstream a distance of about 2,700 feet. Improvements were hydrologically sized for the 2 percent chance, 1 percent chance and SPF levels of protection. The 1 percent chance flow with corresponding ice surcharge effect was eventually selected as the design level of protection through the plan formulation process. Properties protected by this dike are predominantly commercial areas along the Aroostook River as shown on Plate 7.

b. Project Design Flood. Based on scoping analyses during DPR studies, the recommended design flood is the 1 percent chance peak flow of 74,000 cfs plus 1.7 ft. allowance for ice jam surcharge. The water surface elevations for the 1 percent chance flow at the downstream and upstream ends of the dike, without ice surcharage, are 364.0 and 365.7 feet NGVD, respectively, and at the Limestone Road bridge is 366.1 feet



SAINT JOHN RIVER BASIN

AROOSTOOK RIVER

STANDARD PROJECT FLOOD HYDROGRAPH

D.A = 2230 SQ. MI.

FORT FAIRFIELD MAINE

PLATE A-6

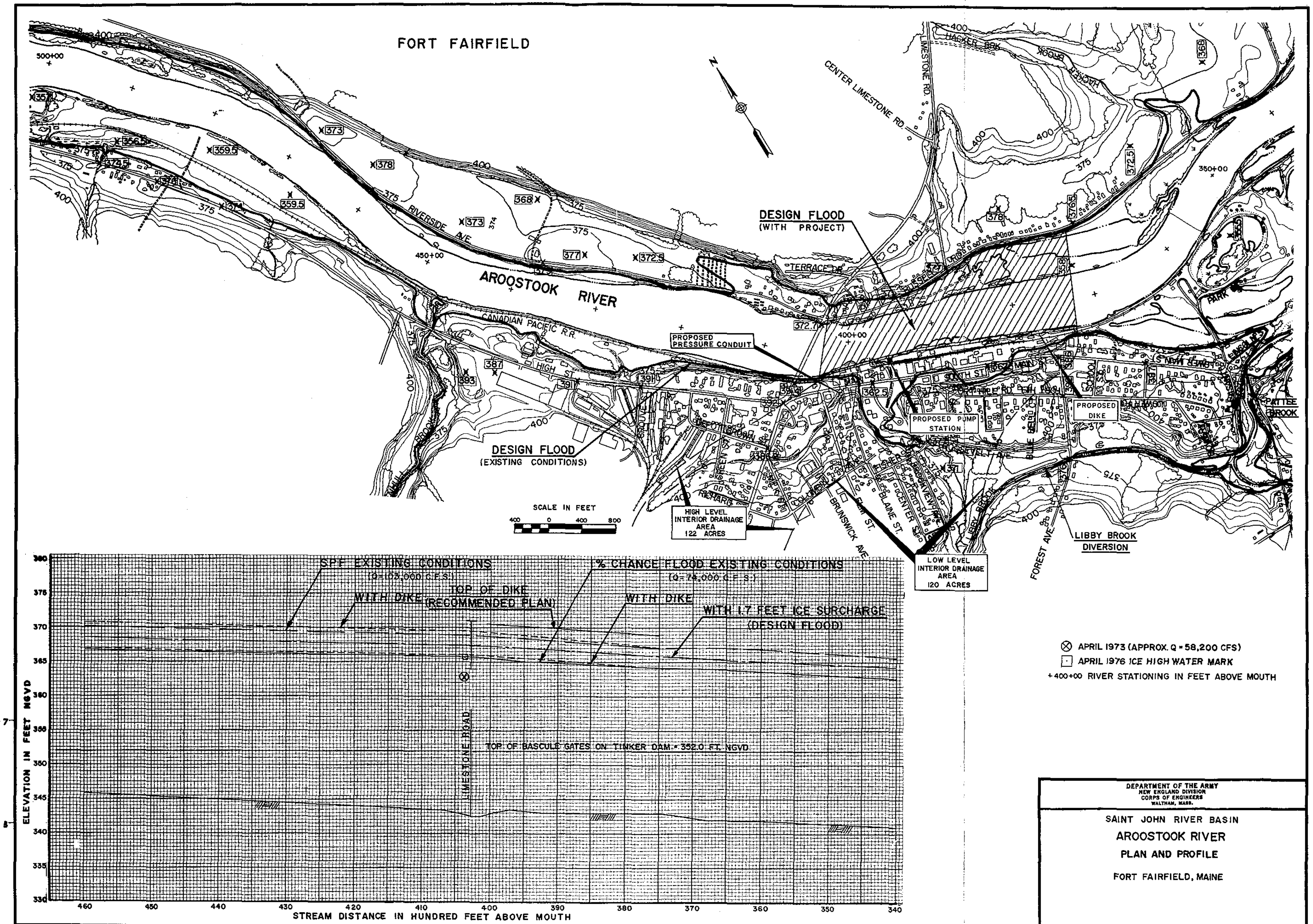

FORT FAIRFIELD
AROOSTOOK RIVER
CANADIAN PACIFIC R.R.
RIVERSIDE AVE
HIGH ST.
DESIGN FLOOD
(WITH PROJECT)
DESIGN FLOOD
(EXISTING CONDITIONS)
PROPOSED PRESSURE CONDUIT
PROPOSED PUMP STATION
PROPOSED DIKE
HIGH LEVEL INTERIOR DRAINAGE AREA 122 ACRES
LOW LEVEL INTERIOR DRAINAGE AREA 120 ACRES
LIBBY BROOK DIVERSION
PATTEE BROOK
CENTER LIMESTONE RD.
LIMESTONE RD.
TERRACE DR.
BRUNSWICK AVE.
FOREST AVE.
SCALE IN FEET
400 0 400 800
SPF EXISTING CONDITIONS
(Q=103,000 C.F.S.)
WITH DIKE
TOP OF DIKE
(RECOMMENDED PLAN)
1% CHANCE FLOOD EXISTING CONDITIONS
(Q=74,000 C.F.S.)
WITH DIKE
WITH 1.7 FEET ICE SURCHARGE
(DESIGN FLOOD)
LIMESTONE ROAD
TOP OF BASCULE GATES ON TINKER DAM = 352.0 FT. NGVD
ELEVATION IN FEET NGVD
STREAM DISTANCE IN HUNDRED FEET ABOVE MOUTH
APRIL 1973 (APPROX. Q = 58,200 CFS)
APRIL 1976 ICE HIGH WATER MARK
+400+00 RIVER STATIONING IN FEET ABOVE MOUTH
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
SAINT JOHN RIVER BASIN
AROOSTOOK RIVER
PLAN AND PROFILE
FORT FAIRFIELD, MAINE

PLATE A-7

NGVD. Water surface elevations were computed by backwater computations using the HEC-2 computer program with a channel Manning's "n" value of 0.03. *Flood flow velocities within this reach ranged from 7 to 11 feet* per second and are not measurably different from those computed without the project. As a result of the project, the Aroostook River would experience an increase in stage at the Limestone Road bridge of about 0.5 feet. Modified discharge rating curves as well as stage frequency curves, reflecting ice surcharge, are shown on Plates 3 and 4, respectively. Plan and profiles of the 1 percent chance flood with and without ice surcharge as well as the standard project flood are shown on Plate 7.

c. Level of Protection. The proposed plan will provide flood damage protection to properties located along the right bank of the Aroostook River. Top elevation of the dike will be 368.7 feet NGVD at its downstream end sloping uniformly to elevation 370.4 feet NGVD at its upstream end. The design elevations for the top of dike will provide 3 feet of freeboard above the ice surcharged 1 percent chance flood (74,000 cfs plus 1.7 feet increase in stage) level and will be 4.7 feet above the April 1976 flood of record at Fort Fairfield. Similarly, the dike height would provide 3 feet of freeboard above a floodflow 85 percent of the 1 percent chance flood with a coincidental 3.4 feet ice jam surcharge. The selected 3 feet of freeboard above the ice surcharged 1 percent chance flood level will provide some allowance for added ice, debris, or other unpredictable surcharge inducing factors during the design event.

d. Riprap Design. All disturbed earth channel side slopes will be riprap protected. Hydraulic analysis for riprap design was provided by the Hydraulics and Water Quality Section, Water Control Branch using tractive force theories in accordance with EM 1110-2-1601 and ETL 1110-2-120. Riprap was sized with an associated flow depth of 21.4 feet, and for an energy gradient of .00076 foot/foot. Assuming a 1V:2H sideslope, a minimum D_{50} stone size of 0.35' was determined to resist tractive forces alone. Rock size and layer thickness will be increased to reduce damage expected from ice attack and eddy forces.

e. Alternative Levels of Protection. The following two alternative levels of protection were investigated during DPR studies but were found to provide less total net benefits than the recommended plan.

(1) Two Percent Chance (50-Year Design). In order to provide two percent chance flood damage protection to properties located along the right bank of the Aroostook River, the top elevation of the dike would be 367.7 feet NGVD at the downstream end sloping uniformly to elevation 369.4 feet NGVD at the upstream end, providing 3 feet of freeboard above the ice surcharged 2 percent chance flood (67,000 cfs) level and would be 3.8 feet above the April 1976 flood of record level at Fort Fairfield. Similarly, the dike height would provide about 3 feet of freeboard above a floodflow 82 percent of the two percent chance flood with a coincidental 3.4 foot ice jam surcharge.

(2) Standard Project Flood Design. To provide standard project flood protection, the top elevation of the dike would be 370.1 feet NGVD at its downstream end sloping uniformly to elevation 371.9 feet NGVD at its upstream end. The design elevations for the top of dike would be 1.4 feet higher than for the one percent chance design (recommended plan) and would provide 3 feet of freeboard above the non-ice standard project flood (103,000 cfs) level and is 6.2 feet above the greatest experienced flood level at Fort Fairfield. The SPF design level was not adopted for ice on the thesis that under SPF conditions any ice would go out prior to the occurrence of peak flow.

f. Interior Drainage.

(1) General. The proposed earth dike will intercept runoff from approximately 242 acres of interior area consisting of residential/-commercial areas and farmlands. The interior area was divided into (a) a "high level" watershed of approximately 122 acres which will discharge by gravity via pressure conduit during periods of high flow on the Aroostook River and (b) a "low level" watershed of approximately 120 acres which will drain by gravity during normal periods but will require pumping during high river stages.

(2) High Level Watershed. The high level watershed is situated on the western side of Fort Fairfield consisting of about 60 percent farmlands and 40 percent residential/commercial areas. Runoff from this area flows northerly along the eastern side of the Bangor and Aroostook Railroad and passes through a series of culverts before outletting into the Aroostook River via a 5-foot diameter corrugated metal pipe. Interior drainage requirements for this high level watershed consist of a 48 inch diameter presssure conduit extending from upstream of Main Street to the Aroostook River for a total length of about 350 feet. Top elevation of the headwall above Main Street will be 372.0 feet NGVD, including 2 feet of freeboard, in order to pass the 1 percent chance discharge of 83 cfs against design river stage. This flow capacity is based on the rational formula using a 1 percent chance 1 hour rainfall of 1.9 inches and a runoff coefficient of 0.36. The upstream invert elevation of the proposed pressure conduit will be 366.0 feet NGVD.

(3) Low Level Watershed. The low level watershed is situated in the central part of Fort Fairfield, consisting of about 85 percent moderate business and residential development and 15 percent un-developed. In the 1960's, a locally constructed channel diverted Libby Brook easterly into Pattee Brook which outlets into the Aroostook River downstream of the proposed line of protection. The remaining undiverted portion of Libby Brook flows through the central part of town and outlets into the Aroostook River through the proposed line of protection via twin 8-foot diameter corrugated metal conduits. Runoff from this low level watershed is conveyed to the Aroostook River by this undiverted portion of Libby Brook. Interior drainage requirements consist of a 48-inch diameter gated gravity conduit, located at the line of protection with capacity to

discharge a minimum of 125 cfs against a normal river stage. The gated gravity conduit should have a slope no less than 0.01 foot per linear foot. This flow capacity is based on the rational formula using a 1 percent chance 1 hour rainfall of 1.9 inches, and a "c" coefficient of 0.55. An interior pumping station will also be required with a capacity of 30 cfs against design river stage. This pumping capacity is equivalent to a runoff rate of 0.25 inch per hour which is comparable to the maximum average hourly rainfall rate during past historic floods, most notably, the April 1973 and April 1983 events. Both the pumping station and gated gravity conduit will be located adjacent to the proposed line of protection just north of Main Street in the vicinity of the present Libby Brook outlet.

SECTION B

GEOTECHNICAL AND DESIGN CONSIDERATIONS

LOCAL PROTECTION PROJECT
ST. JOHN RIVER BASIN
FORT FAIRFIELD, MAINE
GEOTECHNICAL STUDIES

TABLE OF CONTENTS

A. PERTINENT DATA

1. Purpose

   Local flood protection

2. Location

   State - Maine
   County - Aroostook
   City - Fort Fairfield

3. Design Flood

   Frequency - 100-year flood
   Freeboard - 3 feet
   D_{50} - 0.44 feet for 1 vertical to 2 horizontal slope

4. Dike

   Type - Earth Fill with Stone Protection
   Maximum height above streambank - 28 feet
   Maximum height above landside toe - 18 feet
   - 16 feet (alternate)
   Slopes - Riverside - 1 vertical on 2.5 horizontal
   - Landside - 1 vertical on 2.5 horizontal
   Total Length - 3,175 feet
   - 2,730 feet (alternate)
   Top Width - 12 feet to 17 feet (transition sections)

5. Pump Station

   Type - Concrete
   Bottom Elevation - 342 feet NGVD
   Capacity - 30 cubic feet per second

6. Pressure Conduit

   Type - Concrete/Dustile Iron
   Invert Elevation(s) - 366 feet NGVD to 342 feet NGVD
   Diameter - 4 feet

7. Railroad Gates

   Type - Stop log
   Bottom of footing elevation - 354 feet NGVD

B. INTRODUCTION

8. Location and Description of Project

The proposed flood damage reduction project in Fort Fairfield, Maine is situated on the south bank of the Aroostook River. The Aroostook River originates approximately 61 miles to the southwest of Fort Fairfield at the east outlet of Munsungan Lake in Township 8, Range 9, Maine. It flows in a northeastly direction after passing through Fort Fairfield approximately 9 miles to its confluence with the St. John River in Four Falls, New Brunswick, Canada. The project will consist of a 3,175 or 2,730 foot (alternate) foot earth dike situated on the south bank of the Aroostook River, a pump station and pressure conduit to handle interior drainage, and two railroad gates to provide end closures for the dike. The project will reduce flood damage to private and commercial properties in the Fort Fairfield central business district during large flood events.

9. General

Subsurface investigations and geotechnical engineering studies were performed to further the continued planning of structural features to reduce flood damage in Fort Fairfield, Maine. The subsurface investigations included research of available information, geological studies, subsurface explorations and laboratory testing. The subsurface investigations were performed to determine the distribution and description of potential foundation materials for the proposed improvements. Preliminary geotechnical engineering studies, based on the data collected from the subsurface investigations were conducted to develop safe and economical preliminary foundation designs, dike sections, and construction methods.

Additional Plan Formulation was done after completion of subsurface investigations and most of the geotechnical engineering effort for this report. Changes due to the additional plan formulation are designated as "alternate" on the plates and in the text. Subsurface explorations and geotechnical studies will be required during the plans and specifications stage to accommodate the alternate pump station and south gate structure locations.

10. Elevations

All elevations mentioned in this report are in reference to the National Geodetic Vertical Datum (NGVD), which is the mean sea level of 1929.

C. TOPOGRAPHY, GEOLOGY AND SEISMICITY

11. Topography

The project site is on the south bank of the Aroostook River about nine river miles southwest from its confluence with the St. John River in New Brunswick, Canada. The centerline of the proposed dike is along a sloping river bank which averages about 80 feet wide and varies in elevation (El.) from approximately 340 feet to 360 feet. Terraces are well developed on the opposite bank of the river. Away from the river banks, low, rounded hills rise to about El. 700 feet.

12. Geology

The bedrock of the area is mapped as the Spragueville Formation, a calcareous metasiltstone with interbedded silty limestones. Borings along the alignment went to elevations as low as El. 327 feet with none reaching bedrock, but State of Maine Route 165 highway bridge borings reached bedrock as shallow as El. 345 feet where the railroad passes under the highway bridge just upstream of the project site. The borings show that the rock surface plunges as deep as El. 270 feet toward the north bank of the river, or about 75 feet below the water surface. Along the dike alignment the overburden consists of fill, sands and gravels with minor silts which overlie a sandy gravelly till.

13. Seismicity

The project is located in Seismic Zone 1 as defined by the map contained in Engineering Regulation, ER 1110-2-1806, "Earthquake Design and Analysis for Corps of Engineers Projects." A seismic coefficient of 0.05g is to be used for stability analyses of concrete structures.

D. SUBSURFACE INVESTIGATIONS

14. Presentation of Data

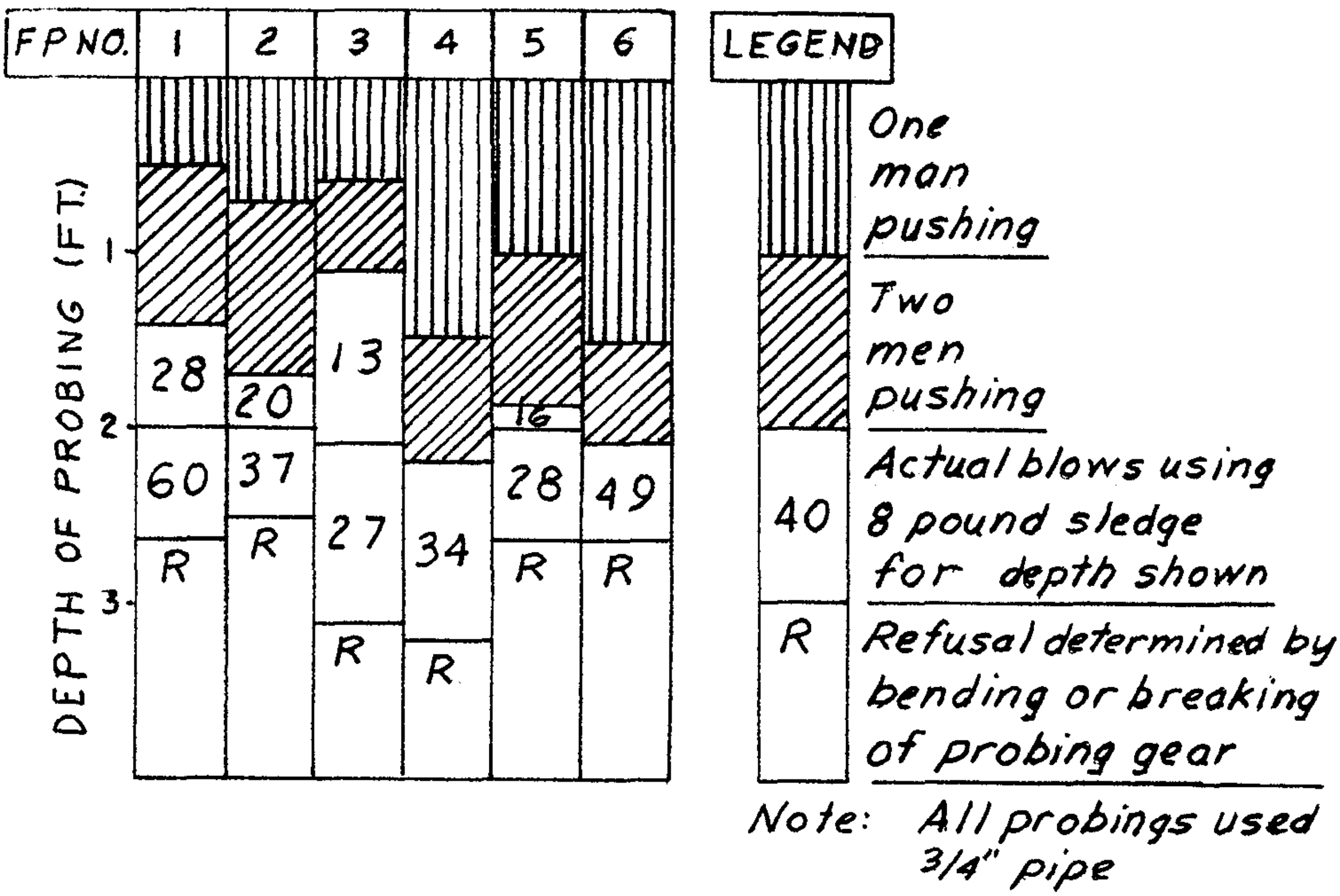
Locations of the subsurface explorations are shown on Plates B-1, B-2, and B-3. An Engineering-Log Profile of the borings is presented on Plate B-4. Probe data is shown on Table B-1. The results of soil tests are included in Table B-2.

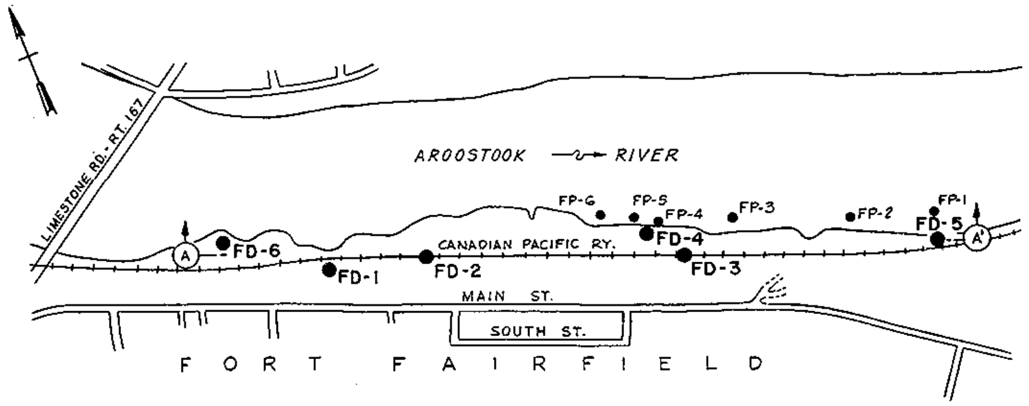
15. Subsurface Explorations

Atlantic Testing Laboratories, Limited executed seven hollow stem auger borings (FD-86-7 to FD-86-13) for the United States Army Corps of Engineers (USACE), March 10-12, 1986. The boreholes were advanced in areas where proposed structures are to be constructed. Standard Penetration Tests and split spoon samples were generally taken at 5-foot intervals or more frequently when required by the inspector. The test borings were terminated at depths from 12 feet to 32 feet.

TABLE B-1

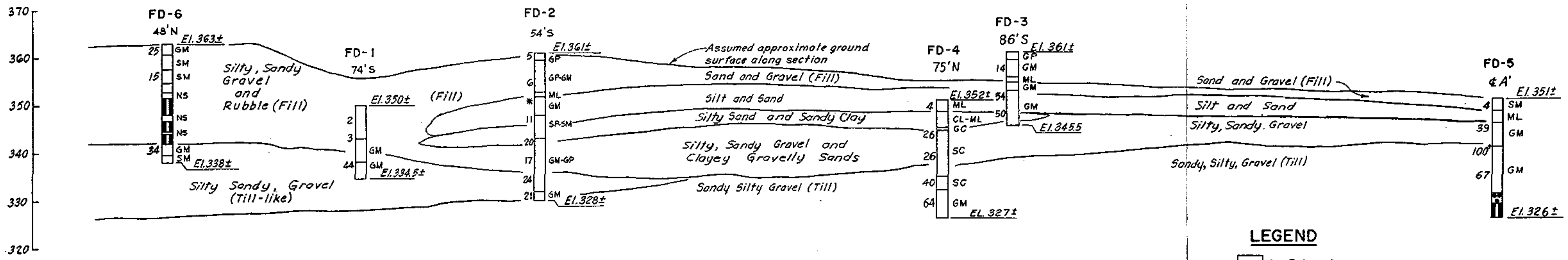
PROBES





BORING PLAN

SCALE IN HUNDREDS OF FEET

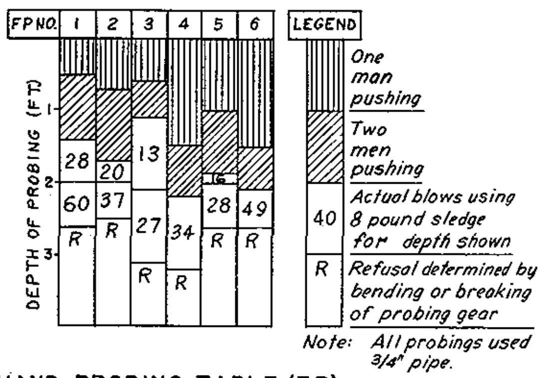


INTERPRETIVE GEOLOGIC SECTION

SCALE: HORIZ. 1" = 100'
VERT. 1" = 10'

LEGEND

- Subsurface water level
- NS Not sampled
- 22 Representative blow count using a 350 lb. hammer with fall of 18 inches
- * Blow count not considered representative
- SM Graphic symbol according to Unified Soil Classification System
- Cobble or boulder (Core drilled)
- Nested boulders or cobbles
- A — A Line of Cross-Section
- FD Foundation drill hole
- FP Foundation probe



HAND PROBING TABLE (FP)

NOTES:
Test borings FD-1 thru FD-3 made by the Corps of Engineers 10-12 July 1950.
Test borings FD-4 thru FD-6 made by the Corps 27 September thru 4 October 1978.
Hand probings were made by the Corps on 6 October 1978.

GRAPHIC SCALES

1" = 10' 10 0 10 20

1" = 100' 100 0 100 200

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN STUDY

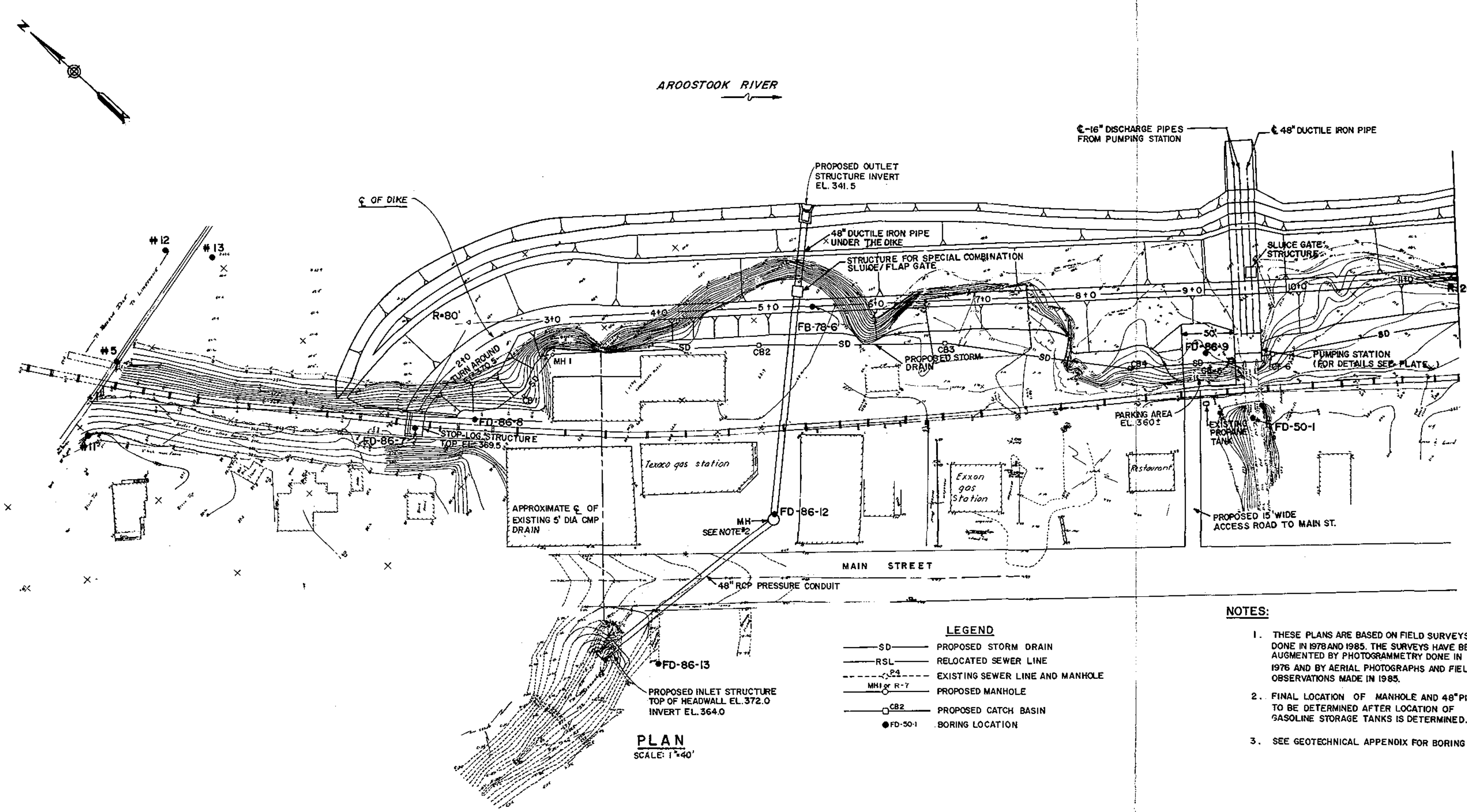
AROOSTOOK RIVER WATERSHED
AT FORT FAIRFIELD, MAINE

BORING AND PROBING DATA

R J D
R J D
C G H

GEOTECH. ENG. BR.
SCALE: AS SHOWN
PLATE I
DATE: 3 MAY 1983

PLATE B-4

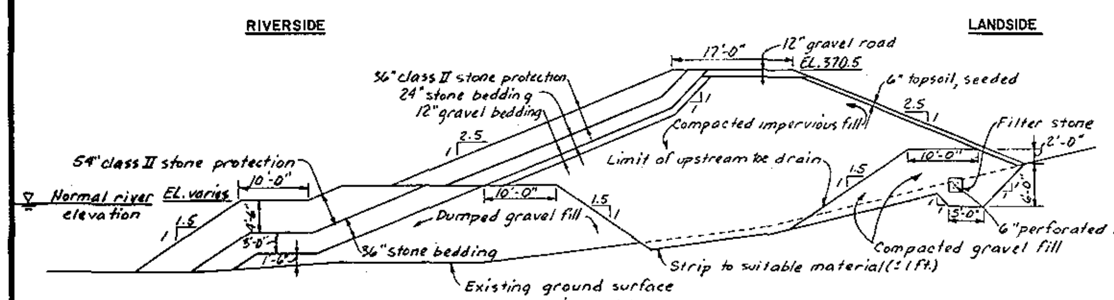


PLAN

SCALE: 1"=40'

NOTES:

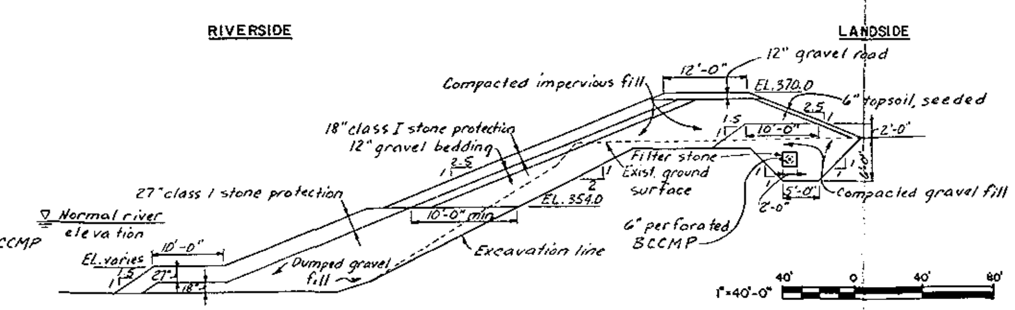
1. THESE PLANS ARE BASED ON FIELD SURVEYS DONE IN 1978 AND 1985. THE SURVEYS HAVE BEEN AUGMENTED BY PHOTOGRAMMETRY DONE IN 1976 AND BY AERIAL PHOTOGRAPHS AND FIELD OBSERVATIONS MADE IN 1985.
2. FINAL LOCATION OF MANHOLE AND 48" PIPE TO BE DETERMINED AFTER LOCATION OF GASOLINE STORAGE TANKS IS DETERMINED.
3. SEE GEOTECHNICAL APPENDIX FOR BORING AND PROBE DATA.



TYPICAL SECTION

FROM STA. 0+00 TO STA. 3+20

SCALE: 1"=10'-0"



TYPICAL SECTION

FROM STA. 3+20 TO STA. 7+50

SCALE: 1"=10'-0"

1"=40'-0" 40' 0 40' 80'

1"=10'-0" 10' 0 10' 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN

LOCAL PROTECTION PROJECT

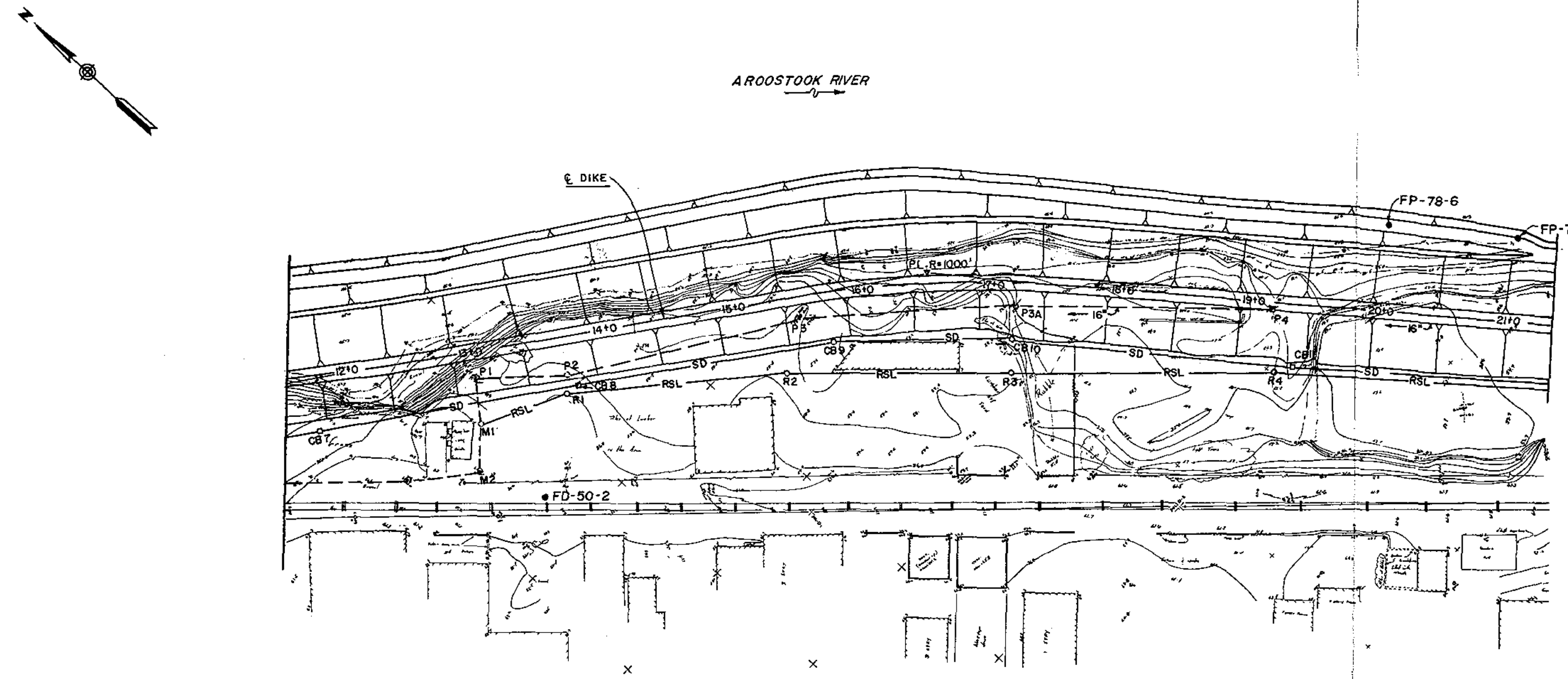
AROOSTOOK RIVER

FORT FAIRFIELD, MAINE

GENERAL PLAN AND SECTIONS NO. 1

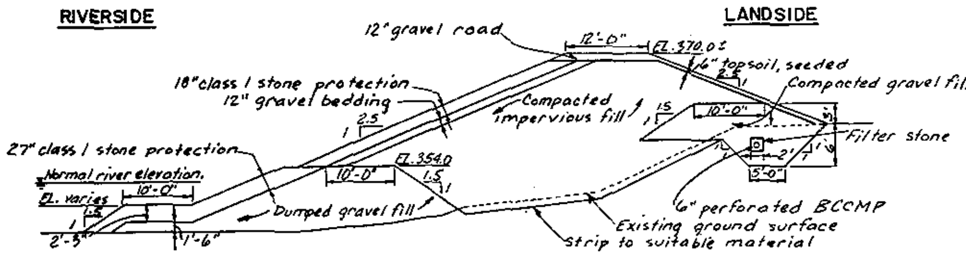
SCALE: AS SHOWN — DATE: MAR. 1987

PLATE B-1

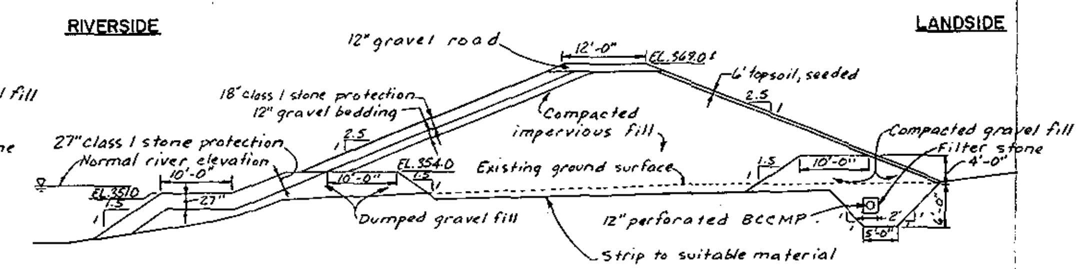


PLAN
SCALE: 1"=40'

NOTE: For notes and legend see plate



TYPICAL SECTION
FROM STA. 7+50 TO STA. 16+00
SCALE: 1"= 10'-0"



TYPICAL SECTION
FROM STA. 16+00 TO STA. 26+00
SCALE: 1"= 10'-0"

1"=40'-0" 40' 0 40' 80'

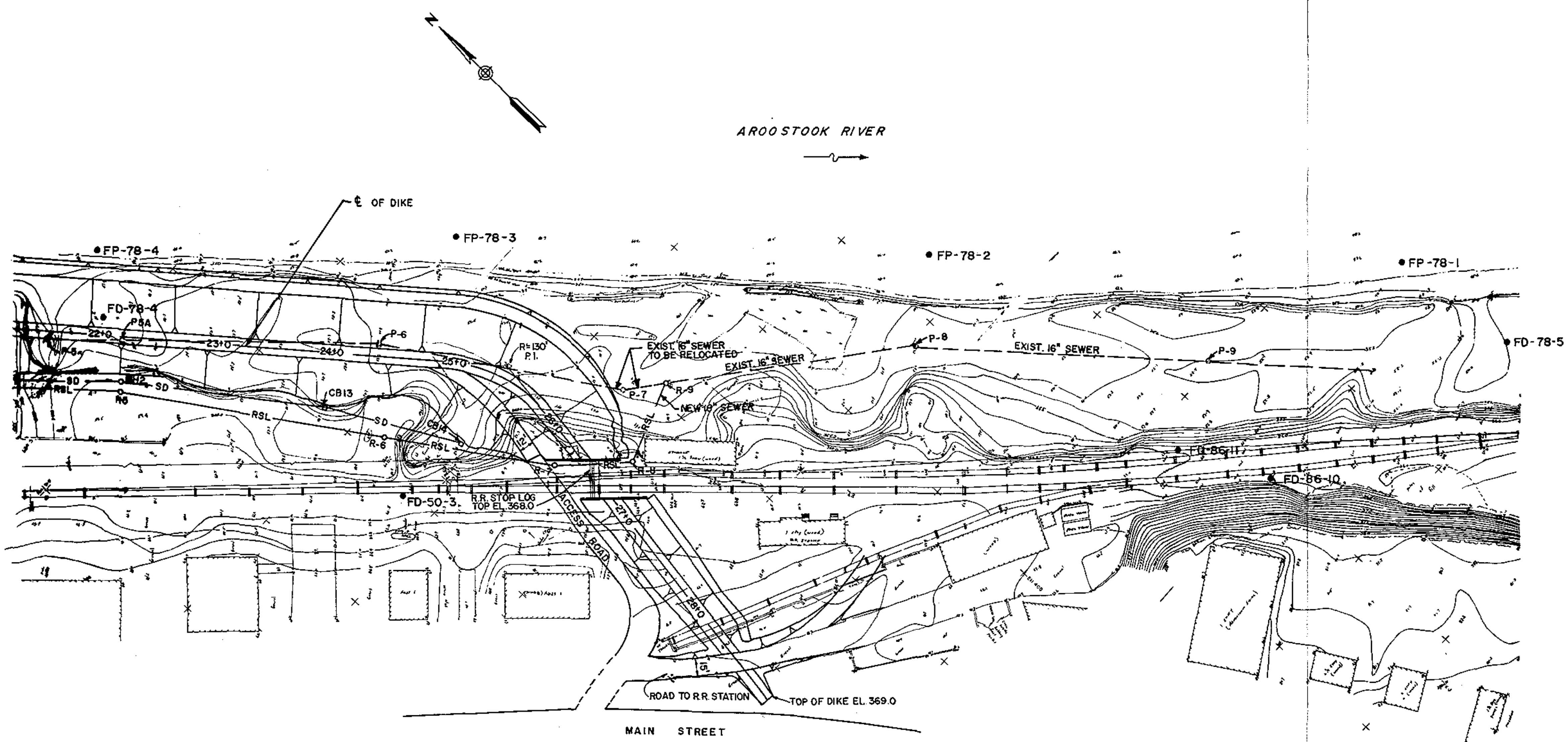
1"=10'-0" 10' 0 10' 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN
LOCAL PROTECTION PROJECT
AROOSTOOK RIVER
FORT FAIRFIELD, MAINE
PLAN AND SECTIONS NO.2

SCALE: AS SHOWN DATE: MAR. 1987

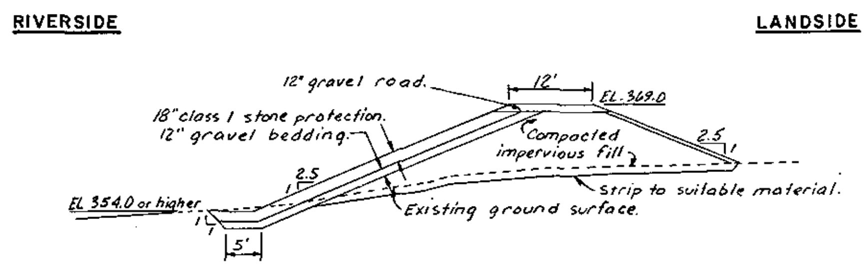
PLATE B-2



PLAN

SCALE: 1"=40'

NOTE.
FOR NOTES AND LEGEND SEE PLATE



TYPICAL SECTION

FROM STA. 26+00 TO END OF DIKE

SCALE: 1"=10'-0"

1"=40'-0" 40' 0 40' 80'

1"=10'-0" 10' 0 10' 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN

LOCAL PROTECTION PROJECT

AROOSTOOK RIVER

FORT FAIRFIELD, MAINE

GENERAL PLAN AND SECTIONS NO. 3

SCALE: AS SHOWN | DATE: MAR. 1987

PLATE B-3

TABLE B-2

SOIL TESTS RESULTS

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT TOTAL	-NO4	COMPACTION DATA STND. AASHO OPT. WATER % DRY WT	MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT TOTAL	-NO4	OTHER TESTS SHEAR	CONSOL.	PERM.
FD-78-4	352.0	J3	2.7-5.0	CL-ML	0	40	60	0.003	27	21		35.5									
FD-78-4	352.0	J7-1	6.7-10.2	CL	0	30	70	0.005	31	22		27.9									
FD-78 -4	352.0	J7-2	6.7-10.2	CL	0	0	96	<0.001	27	19		22.2									
FD-78 -4	352.0	J13	15.0-16.2	SC	5	65	30	0.015													
FD-78-4	352.0	J14	16.2-19.4	SC	38	40	22	0.025													
FD-78-4	352.0	J16	20.0-24.5	GC	55	30	15	0.03													
FD-78-5	351.0	J 7	5.6-10.0	GP-GM	59	34	7	0.3													
FD-86-7	361.0	S-3	10.0-12.0	SW-SM	24	69	7	0.2													
FD-86-8	361.0	S-3B	11.5-12.0	SM	1	67	32	-													
FD-86-8	361.0	S-4	15.0-17.0	SM	1	71	28	-													
FD-86-9	363.0	S-4	15.0-17.0	GM	47	40	13	-													
FD-86-10	361.0	S-2	5.0-7.0	SM	22	63	13	-													

* PROVIDENCE VIBRATED DENSITY TEST

Three drive sample borings (FD-78-4 to FD-78-6) and six hand probes (FP-78-1 to FP-78-6) were performed and inspected by the USACE from September 27, 1978 to October 6, 1978. The borings were terminated at 25 feet of depth along the centerline of the proposed dike. Continuous sampling was performed in the boreholes by driving 2-1/2-inch and 2-inch inside diameter solid spoons with a 350 pound weight and 18-inch drop except where diamond core drilling was required to penetrate obstructions. The hand probes were advanced near the normal Aroostook River water line with an eight pound sledge to depths from 2.6 feet to 3.2 feet.

The Maine State Highway Commission performed 13 drive sample explorations for the Aroostook River bridge which is approximately 500 feet north (upstream) of the proposed dike. The explorations varied in depth from 20.0 feet to 88.8 feet. Solid tube samples were generally taken at 5-foot intervals. Rock was cored in 12 of the holes.

Three preliminary drive sample borings (FD-50-1 to FD-50-3) were executed and inspected by USACE, July 10-12, 1950. The borings were located along the existing Canadian Pacific Railroad main line track which is from 90 feet to 175 feet west (inland) of the proposed dike. The depth of the borings varied from 15.5 feet to 31.0 feet. Continuous sampling was performed in the boreholes by driving 2-1/2-inch and 2-inch inside diameter solid spoons with a 350 pound hammer and 18-inch drop.

16. Future Explorations

The south gate structure and pump house will be moved to the alternate locations shown on Plates B-1 and B-3. It is recommended that explorations be performed at their alternate locations during plans and specifications stage to identify the depth of firm undisturbed natural materials. It also is recommended that test pits be executed during plans and specifications stage to better define the extent of the rubble fill near FD-78-6, the soft clayey silts, sands, and gravels near FD-78-4, and the location of utility lines.

17. Laboratory Tests

All laboratory tests were performed in accordance with the procedures described in Corps of Engineers Manual EM 1110-2-1906, "Laboratory Soils Testing." All soil samples were visually classified in accordance with the Unified Soil Classification System. Grain size analyses, Atterberg Limit determinations, Hydrometer analyses, and Moisture Content determinations were performed on selected samples to help classify the materials encountered and to provide more precise data where required.

E. CHARACTERISTICS OF FOUNDATION MATERIALS

18. Dike

Most of the riverside toe of the dike will lie in the pool (normal water El. 352 feet) created by Tinker Dam which is located approximately one mile downstream. The six probes taken in the proposed toe area indicate that the depth to firm ground is 12 to 18 inches. The soil profile under the proposed dike, is granular fill underlain by silty sandy gravels (GM) and silty gravelly sands (SM). Exceptions to the profile were observed near FD-78-6 where rubble fill was encountered and FD-78-4 where clayey silts, sands, and gravels were observed beneath the fill.

The fill is a brown to dark brown, heterogeneous mixture of silt, sand and gravel with cinders, organic matter, brick fragments, porcelain fragments, glass, roots, concrete, tar paper, steel rods, cobbles, boulders and sometimes having an organic odor. The observed thickness of the fill varies from 1.5 feet to 22.0 feet. Blow counts recorded during standard penetration tests and solid spoon drives indicate the fill is very loose to very compact.

Light brown, brown and gray-brown silty sandy gravels (GM) and silty gravelly sands (SM) were observed below the fill. The silt content varied from 7 to 32 percent in grain size determinations performed on the silty sandy gravels and silty gravelly sands. Standard Penetration test and solid spoon sample blow counts indicate the silty sandy gravels and silty gravelly sands are very loose to very compact. Most of the very loose materials are near the top of the silty sandy gravel and silty gravelly sand layer.

19. Gate Structures and Pump Station

The soil profile beneath the gates structures and pump station is similar to the one beneath the dike. The fill thickness is 5.0 feet to 11.5 feet at the proposed north gate structure, 15.0 feet at the proposed pump station, and 0 feet to 5.0 feet at the proposed south gate structure.

20. Pressure Conduit

The soil profile along the proposed pressure conduit is granular fill underlain by a brown, sandy silt. The granular fill varies 0 feet to 4.0 feet in thickness and is similar to the fill material below the proposed dike embankment. The brown, sandy silt is nonplastic. It is loose to moderately compact in consistency based on standard penetration test results.

21. Groundwater

Groundwater was encountered in the boreholes from El. 348 feet to El. 351 feet except for FD-78-6 and FD-86-13 where none was observed, FD-86-11 (El. 343 feet), and FD-86-12 (El. 361 feet). It must be noted that fluctuations in the groundwater levels may occur because of variation in rainfall, snow, ice, temperature, or other factors which differ from the conditions present at the time the observations were made.

22. Shear Strength and Permeability

Shear strength and permeability tests were not performed on the foundation soils. The estimated angle of internal friction for the foundation soils is 28 to 30 degrees. The estimated coefficient of vertical permeability for the foundation soils is (0.3 to 3) x 10-4 cm/s. The estimates are based on visual examination of the samples, grain-size distribution curves, data from exploration logs and experience with similar materials.

23. Consolidation

Consolidation tests were not performed on samples of foundation soils. All soft and compressible surficial materials will be removed prior to the construction of the dike embankment. The consolidation characteristics and natural densities of the principally granular foundation soils beneath the surficial materials are such that significant post-construction foundation settlement is not anticipated under the proposed embankment loadings.

F. CHARACTERISTICS OF EMBANKMENT MATERIALS

24. General

Most of the materials from the required stripping and excavation operations will not be suitable for use in construction of the dike embankment. The suitable material from the excavation and stripping operations will be used to the extent practicable. The contractor will furnish all embankment materials other than those available from the required excavation and stripping operations due to the high cost of developing government furnished borrow areas and difficulty involved in acquiring the land for borrow areas.

25. Filter Design

The gradation requirements for impervious fill, gravel bedding, stone bedding, and stone protection have been established in accordance with the filter criteria set forth in Engineering Manual, EM 1110-2-1913, "Design and Construction of Levees."

26. Impervious Fill

Impervious fill will be furnished by the contractor. It will be a natural, reasonably well graded, unprocessed material which contains clay, silt, and sand. Experience with materials meeting the gradation ranges below indicates that placement moisture contents can be maintained within two percent of optimum moisture content with moderate control and that in-place dry densities will be approximately 135 pounds per cubic foot.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	100
3-inch	85-100
No. 4	70-95
No. 40	35-70
No. 200	20-45

27. Gravel Bedding

Gravel Bedding will be furnished by the contractor. It shall consist of tough, durable particles of sand and gravel or crushed stone which are reasonable well rounded. The materials shall be reasonably well graded within the limits specified below.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	100
1-inch	50-90
No. 4	25-75
No. 16	15-50
No. 200	0-5

(In addition, not more than 10 percent, by dry weight, of the component passing the No. 4 sieve shall pass the No. 200 sieve.)

28. Stone Bedding

Stone bedding will be furnished by the contractor. It shall consist of quarried rock, composed of hard, durable, angular and sound rock fragments. Stone bedding shall be reasonably well graded within the limits specified below.

Sieve Size (U.S. Std.)	Percent Passing by Dry Weight
6-inch	90-100
1-1/2 inch	0-40
No. 4	0-5

29. Stone Protection

Stone protection will be furnished by the contractor. It shall consist of quarried rock, composed of hard durable, angular and sound rock fragments with a unit weight of not less than 162 pounds per cubic foot. It shall meet the following gradation and size requirements.

Class	Limits of Stone Weight (Pounds)	Percent Lighter by Weight
I	Between 120 and 300 (Max)	100
	Between 60 and 90	50
	Less than 20	15
	2 (Min.)	0
II	Between 900 and 2300 (Max.)	100
	Between 450 and 700	50
	Less than 150	15
	2 (Min.)	0

30. Shear Strength and Permeability

It is estimated based on the above gradations that the proposed embankment materials will develop the following angles of internal friction and coefficients of permeability:

Materials	Angle of Internal Friction (Degrees)	Coefficient of Permeability (cm/s)
Compacted Impervious	30 + 32	$<10^{-4}$
Dumped Gravel	30 to 33	10^{-3} to 10^{-2}
Compacted Gravel	35 to 37	10^{-3} to 10^{-2}
Gravel Bedding	35 to 37	10^{-3} to 10^{-2}
Stone Bedding	40	$>10^{-2}$
Stone Protection	40	$>10^{-2}$

31. Sources

Sand, gravel, and stone could be supplied by a commercial supplier in Presque Isle which is approximately 10 miles from the proposed project site. Private sand, gravel, and stone sources exist along the Aroostook River within 5 miles of the project which have been opened for use on past projects. Concrete is available from the suppliers in Presque Isle, Houlton, and Madawaska which are all within 40 miles of the site.

G. DESIGN AND CONSTRUCTION

32. Design Criteria

The principles and procedures discussed in Engineering Manual, EM 1110-2-1913, "Design and Construction of Levees," were used to develop dike sections for this project. Layer thicknesses and stone sizes for the proposed stone protection on the dike were determined using procedures in the Engineering Manual, EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" and Engineering Technical Letter, ER 1110-2-120, "Additional Guidance for Riprap Channel Protection."

33. Materials for Dike Construction

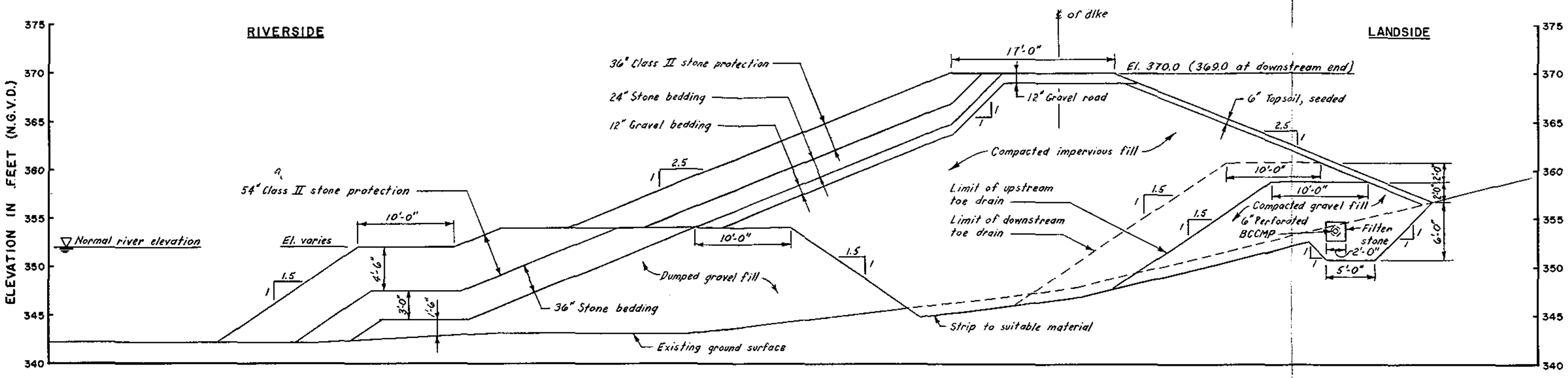
All dike materials will be furnished by the contractor. It is estimated that approximately 2,000 cubic yards of excavation will be required to remove unsatisfactory dike foundation materials. Most of the material excavated will not meet the specifications for the dike embankment materials. The Contractor will be required to dispose of the excavated material that can not be reused at an appropriate upland site.

34. Dike Sections

Proposed dike sections are shown on Plates B-5 and B-6. The shape of the sections was influenced by foundation conditions, seepage control requirements, river erosion, ice action, maintenance considerations, and construction sequence. The stone protection thickness is greater in Section A-A (Typical End Section) than the other sections to reduce erosion caused by eddy currents and ice action at the ends of the dike. The toe on the landside of the dike will interrupt seepage in critical areas and act as an inspection trench during construction. Stone will protect the dike from erosion and ice action on the riverside. Grass, placed at a 1 vertical on 2.5 horizontal slope for maintenance reasons, will protect the landside dike slope. The dumped gravel fill riverside berm will expedite construction of the dike and will allow the contractor to dewater the central dike base prior to placing the compacted impervious fill core. The compacted impervious fill core will cut off seepage.

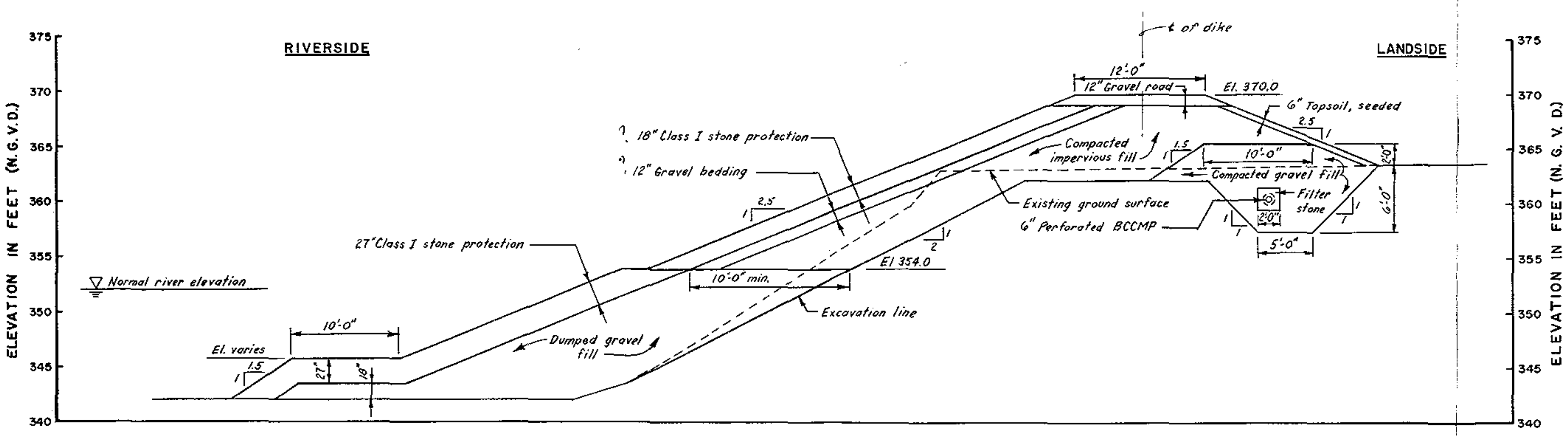
35. Seepage Control

The design hydrostatic head for the dike is the difference between the 100-year flood level (El. 366 feet to El. 367 feet) on the waterside and a water level at the ground surface on the landside. The design hydrostatic head ranges from approximately 3 feet to approximately 15 feet. Seepage through the dike will be controlled by the relatively long seepage path through the impervious core. Foundation seepage will be controlled by the relatively long seepage path through the predominantly silty sandy gravel and silty gravelly sand foundation soils. A shallow landside toe drain will provided to interrupt seepage and reduce softening on the inside of the dike.



SECTION A-A

TYPICAL FROM STA. 0+00 TO STA. 3+00 AND STA. 28+75 TO STA. 31+75



SECTION B-B

TYPICAL FROM STA. 3+00 TO STA. 7+50

NOTES:

1. Dike sections are designed for 100-Year Flood Level and include 3-feet freeboard.
2. All pipes shall be removed from beneath the dike.
3. Existing riverbank slopes shall be cut to 1 vertical on 2 horizontal or flatter.
4. Assume 1 foot of stripping to the Landside of the dumped gravel fill.
5. Assume 2000 cubic yards of trash and debris will be excavated and disposed of off site, in addition to the stripping.
6. Top of bascule gate at Tinker Dam is located approximately 1 mile downstream, is El. 352.0.

GRAPHIC SCALE
1" = 5' 5' 0 5' 10'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
FORT FAIRFIELD, MAINE

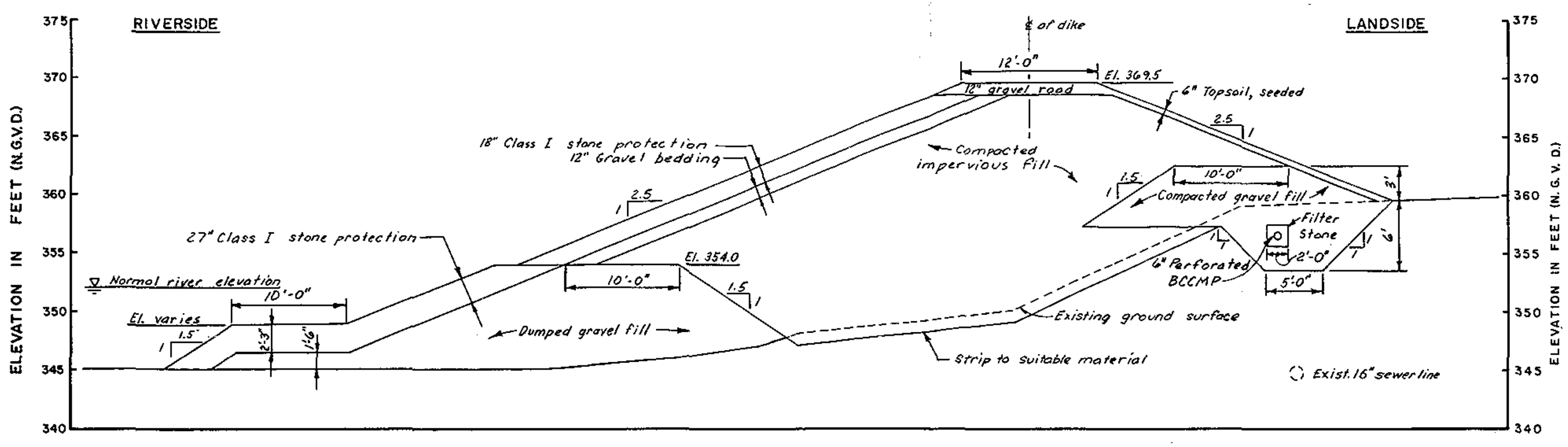
LOCAL PROTECTION PROJECT

TYPICAL SECTIONS NO. 1

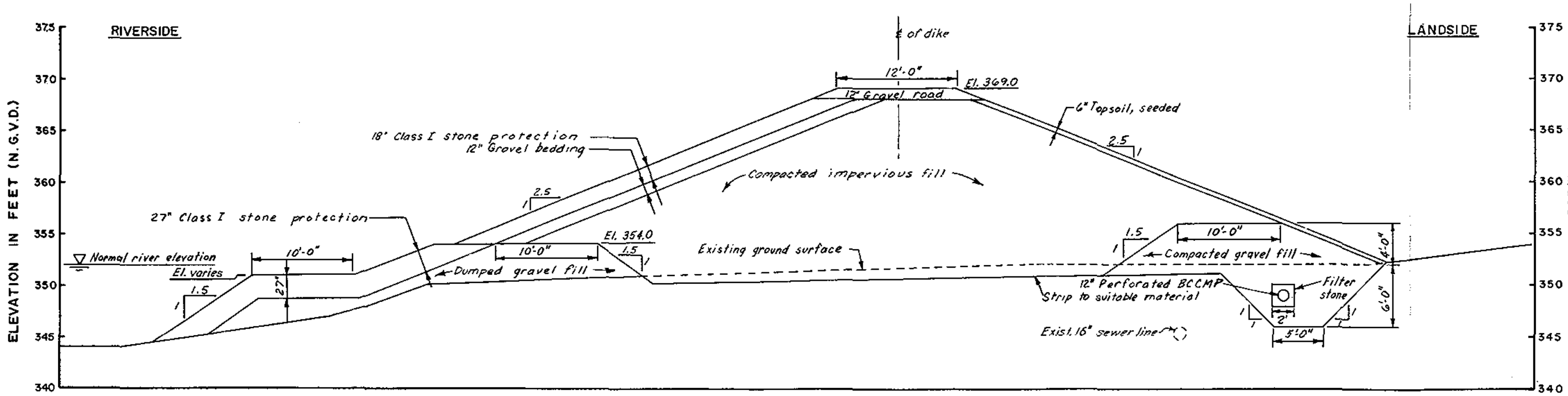
AROOSTOOK RIVER MAINE

PLATE B-5

NOTE:
Notes are shown on PLATE B-5



SECTION C-C
TYPICAL FROM STA. 7+50 TO STA. 16+00



SECTION D-D
TYPICAL FROM STA. 16+00 TO STA. 28+75

GRAPHIC SCALE
1" = 5'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
FORT FAIRFIELD, MAINE

LOCAL PROTECTION PROJECT

TYPICAL SECTIONS NO. 2

AROOSTOOK RIVER MAINE

36. Embankment Stability

Section D-D was selected for stability analysis because it combines maximum embankment height with average to low foundation strengths. Section D-D was analyzed for stability against shear failure using circular failure surfaces and the UTEXAS2 slope stability package for the End of Construction, Sudden Drawdown from Maximum Pool, Intermediate Flood Stage, Steady Seepage from Maximum Pool conditions. An analysis of earthquake conditions was not judged necessary due to the height of the dike, the low magnitude of earthquakes that have occurred in the vicinity of the site in the past, and the characteristics of the dike materials. The design unit weights and shear strength parameters were selected on the basis of experience with similar materials on other projects and are tabulated below:

Material	Unit Weight (pcf)		Shear Strength (degrees, psf)		
	saturated	moist	Q	R	S
Stone Protection	135	118	40,0	40,0	40,0
Gravel Bedding and Compacted Gravel Fill	145	135	35,0	37,0	37,0
Dumped Gravel Fill	135	120	30,0	33,0	33,0
Compacted Impervious Fill	140	135	30,0	30,0	32,0
Foundation Soils (above El. 342.0 feet)	137	130	28,0	28,0	30,0
Foundation Soils (Below El. 342.0 feet)	140	133	30,0	30,0	32,0

The minimum factor of safety for each condition is shown below. The results indicate that the selected embankment is safe from shear failure.

Condition	Factor of Safety Acceptable	Calculated (Shallow)	Calculated (Deep)
End of Construction (Riverside)	1.3	1.6	1.5
End of Construction (Landside)	1.3	1.4	2.0
Sudden Drawdown from Maximum Pool (El. 367)	1.0	1.3	1.2
Intermediate Flood Stage (El. 360 and El. 356)	1.4	1.6	1.6
Steady Seepage from Maximum Pool (El. 367)	1.4	1.5	1.7

37. Dike Settlement

The embankment and foundation soils are of low compressibility except possibly for the rubble fill near FD-78-6 and the clayey silts, sands, and gravels near FD-78-4. The rubble fill and surficial, soft, clayey silts, sands and gravels will be removed prior to construction of the dike. The remaining clayey silts, sands and gravels are judged to be of low compressibilty in situ due to their high densities and low plasticity indices. Therefore, it is expected that all significant settlement of the principally granular embankment and foundation soils will occur during construction.

38. Construction Sequence

The dumped gravel fill riverside toe will be constructed starting at the upstream end by pushing material into and down the Aroostook River with bulldozers. The riverside toe will act as a cofferdam and will facilitate dewatering of the compacted fill areas by open pumping. Deleterious materials will be stripped in the compacted fill areas after completion of dewatering and prior to placement of fills. Compacted fills will be placed to their full width in reaches long enough to permit proper operation of compaction equipment. Stone protection and bedding layers will be placed below normal water without diversion or dewatering of the construction area immediately after completion of the dumped gravel fill riverside toe. Above normal water, they will be placed in the dry after completion of the compacted fills. Dike reaches will be completed to their full width including stone protection prior to flood season.

39. Placement and Compaction

Compacted gravel and impervious fill materials will be spread with bulldozers or other approved equipment in loose layers of 8 inches in non-restricted areas and 4 inches in restricted areas. Each layer will be compacted to 95 percent of its maximum dry unit weight as determined by modified proctor test ASTM D-1557. Heavy tractors and vibratory rollers will not be allowed in restricted areas.

40. Slope Protection

Hydraulic analysis for erosion control of the dike indicates that a minimum D_{50} stone size of 0.44 feet is adequate to resist tractive forces for a 1 vertical to 2 horizontal slope. A stone layer thickness of 0.75 feet was calculated from the minimum D_{50} stone size. The stone layer thickness was increased to 1.5 feet for placement above normal water to resist ice forces, and to 2.25 feet for placement below normal water to resist ice forces and to provide for uncertainties associated with underwater placement. The stone sizes required to construct layers 1.5 feet and 2.25 feet thick will be large enough to be considered vandal proof.

Experience with ice action at Fort Kent, Maine has shown embankment displacements occur in the transition areas even when twice the minimum D_{50} stone size is used to determine the layer thickness. Three times the minimum D_{50} stone size was used to calculate a stone layer thickness of 2.25 feet in the transition areas. The stone layer thickness in the transition areas were increased to 3.0 feet for placement above normal water to resist ice forces, and to 4.5 feet for placement below normal water to resist ice action and to provide for uncertainties associated with underwater placement.

The proposed classes and gradations for the stone protection are listed in Section 29. The proposed stone protection sections are shown on Plates B-5 and B-6.

41. Structures

A pump station, pressure conduit and two railroad gates will be appurtenant structures to the dike. They will be light weight structures constructed at the locations shown on Plates B-1 to B-3. They will be constructed on undisturbed natural soils or compacted gravel fill placed on undisturbed natural soils, and at least 6 feet below grade for adequate frost protection. The proposed bottom elevations for the structures are 354 feet for the gates, 348 feet for the pump station, and from 366 feet to 342 feet for the pressure conduit.

A design bearing pressure of 4000 pounds per square foot will be used to design the spread footings required for the gates and pump house. Design bearing pressures for footings less than three feet in minimum dimension will be reduced to B/3 times the recommended bearing pressure, where B is the smallest dimension of the footing in feet. A minimum width of 18 inches will be maintained for continuous footings.

42. Environmental

The environmental concerns identified to date are: movement of pesticides in the river bottom sediments during construction of the river side toe, disposal of stripped material and rubble fill, migration of fines downstream during the dewatering operation, and a petroleum odor in exploration FD-86-11. The results of an Impact Analysis Branch Sampling and Testing Program conducted during the winter and spring of 1986 indicate the levels of pesticides are not high enough in the river bottom sediments at the site to be concerned that significant amounts will move during construction. It is recommended that additional testing be performed during construction to insure pesticide movement is minimal. The town of Fort Fairfield and the state of Maine will identify appropriate disposal areas for the stripped material and rubble fill. Silt curtains or an alternative will be used to reduce migration of fines downstream during the dewatering operation. The downstream end of the dike will be moved to avoid possible contaminated materials in the vicinity of exploration FD-86-11.

43. Access

A gravel surface access road will run along the crown of the dike to allow for inspection, maintenance, recreation and flood-fighting activities. Either two access ramps and one turnout or one access ramp, one turnout, and turnaround will be provided to facilitate use of the access road. Locations for the access ramps, turnout, and turnaround will be decided during the plans and specifications stage.

44. Pipelines

One 16-inch sewer main and many smaller live and abandoned utility pipes exist under the proposed dike alinement. The sewer main and line utility lines will be moved outside the dike limits. The abandoned utility pipes will be removed prior to construction of the dike. The inspection trench and test pits will be used to search for lines that may not have been identified.

SECTION C

STRUCTURAL DESIGN

DETAILED PROJECT REPORT
FORT FAIRFIELD LOCAL PROTECTION
FORT FAIRFIELD, MAINE

STRUCTURAL DESIGN REPORT

1. Purpose: The purpose of this report is to facilitate the review by a higher authority of the structural design features of the Fort Fairfield Local Protection Project, Fort Fairfield, Maine. This information is presented for inclusion in the Detailed Project Report, prepared under the special continuing authority of Section 205 of the 1948 Flood Control Act, as amended.

2. Introduction: This section presents the criteria, data, and assumptions used for the structural design of the proposed structur. for this project. A brief description of each structure is provided and followed by stability computations designed to investigate the critical design condition.

3. Criteria Documents: Structural design criteria are contained in the publications listed below:

CORPS OF ENGINEER PUBLICATIONS

EC 1110-2-510	"Working draft of the Retaining and Flood Wall Manual", 31 August 1983 with Changes 15 July 1985.
ETL 1110-2-256	"Sliding Stability of Concrete Structures" 24 Jun 198.
EM 1110-2-2501	"Flood Wall Manual" January 1948

4. MAJOR STRUCTURAL FEATURES:

The project involves includes two concrete stop-log structures, a pumping station and a 4-foot diameter Ductile Iron pressure conduit. The pressure conduit requires an inlet headwall, an outlet structure, and an emergency gate well. The gravity discharge conduit at the pumping station also requires a sluice gate well.

The following structures were analyzed for stability:

	Appendix A
a. Stop-log structure / Downstream	page 1
b. Stop-log structure / Upstream	page 9
c. Pressure-conduit Inlet headwall	page 15
d. Pumping station	page 19

Each set of stability computations provides a description of the structure, the design criteria and parameter values used in the calculations. The final design concepts of all the structural features of the job are presented in Plates (C1 - C4) of this report.

5. <u>WORK TO BE COMPLETED</u>: During the preparation of plans and specifications, the detailing of all watertight joints, reinforcemen and equipment installation can be designed.

Special attention should be given to the design of the 48"ø Ductile Iron pressure conduit. At any point under the main body of the dike, the conduit is under about a 29-foot head during an extreme flood condition. Pipe connections should be designed to withstand this pressure. An emergency gate well with a slide-type flap gate was designed to prevent excessive pressures resulting due to a river back flow condition. The flap gate would also allow for the interior drainage operation of the conduit during a flood condition.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG - STRUCTURE / DOWNSTREAM
COMPUTED BY: E Nestorides CHECKED BY: DATE: 2-6-87

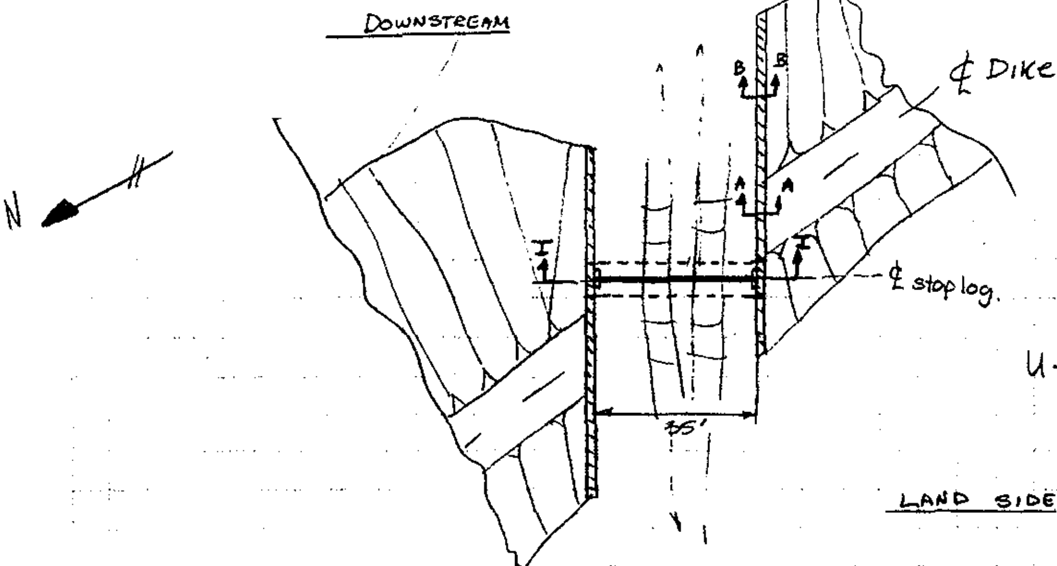
FORT FAIRFIELD - STOP LOG STRUCTURE / DOWNSTREAM

The stop log structure is a concrete U-channel with a center post and 2 bays of stop logs. The opening is about 35 feet wide and positioned on a skew with respect to the dike center line. On either end of the stop log structure concrete gravity walls extend to retain the dike embankment [See plan below].

The structures primarily function as a retaining wall and was designed in accordance to the following Corps Design Criteria:

1. EC 1110-2-510: "Working Draft of the Retaining and Flood Wall Manual" 31 August 1983 (w/changes 15 July '85)
2. EM 1110-2-2501: "Flood Wall Manual" January 1948 (w/changes
3. ETL 1110-2-256: "Sliding stability of Concrete Structures" 24 June

The stop log structures and adjoining walls are shown in plan below:



North wall → 74 ft long
South wall → 56 ft
U-Channel → 8 ft

There will be no sheet pile cutoff under the stop log structure nor a cut-off wall into the dike due to the low differential head and the long line of creep. $\Delta h = 5ft$

Creep Ratio: along wall $\frac{57ft}{5} \Rightarrow 11.4 > 4$ (permissible for sands gravels) OK ✓

under stop logs. $\frac{22}{5} \Rightarrow 4.4 > 4$ OK ✓

DESIGN METHOD

One section of the concrete U-channel and two typical sections of the gravity wall will be analyzed for stability under one load case. Load Case R2 was used for each section since it represents a rapid drawdown situation which is an extreme loading for retaining structure. Since the area in question is in seismic zone one (minor damage) the earthquake condition is not critical.

All the structures are founded on compacted gravel fill and under Load Case R2, they must satisfy the following stability criteria:

a) 75% of the base must be in compression,
b) the factor of safety against sliding must be greater than 1.33,
and c) bearing pressures should not exceed the allowable $4^{k}/ft^{2}$.

Design & Soil Parameters:

	γ_{moist} (kcf)	$\gamma_{saturated}$ (kcf)	$\gamma_{submerged}$ (kcf)	ϕ	assume c / δ
Dumped Gravel bedding (RR)	.120	.135	.073	32°	0 / 0°
Compacted Impervious	.135	.140	.078	30°	0/0°
foundation soil	.130	.137	.075	28°	0/0°

- Allowable bearing capacity for foundation $2^{T}/ft^{2} \Rightarrow 4^{K}/ft^{2}$ [GEB]
- Min. Frost Penetration depth required ⇒ 6 feet
- Consider soil pressures using K_o (at rest)

$$K_o = 1 - \sin\phi \text{ (Jaky's formula)} \quad \phi = 30^\circ \qquad K_o = 1 - .5 \Rightarrow \boxed{K_o = .5}$$

- for sliding:

$$\mu = \tan\phi \Rightarrow \mu = .53 \qquad \phi = 28^\circ \quad c = 0$$

- $\gamma_w = 62.5$ pcf.
- RAPID DRAWDOWN CONDITION: This condition was modeled by treating 60% of the unbalanced fill as submerged and the remaining 40% as saturated.

DESIGN SECTIONS:

The design sections will be on separate plates accompanying this report.

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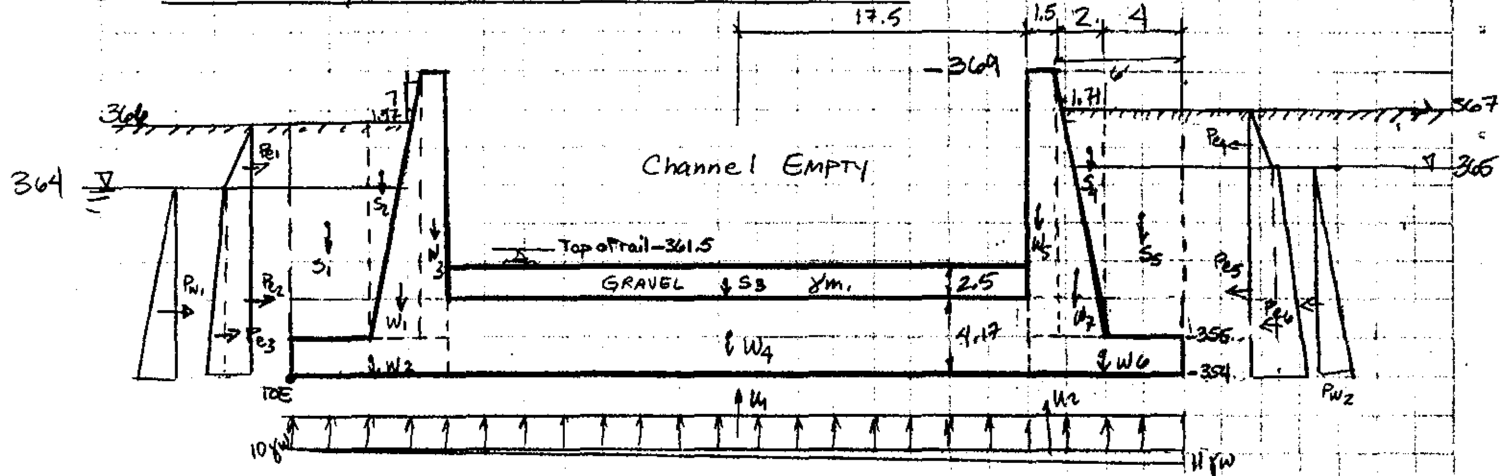
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE / DOWNSTREAM
COMPUTED BY: ENestorides CHECKED BY: DATE: 2/6-87

I. Stop Log Structure → Section I-I:

Not to Scale



Waterlevel was determined by assuming 60% of unbalanced fill as submerged and remaining 40% saturated. Ground level is at 361.

Item	COMPACTION	VERTICAL FORCES	Horizontal Forces	Moment arm	Moment (+)
W_1	½(14.0)(2.0)(.15)	2.10		5.33	11.19
W_2	(7.5)(1.0)(.15)	1.13		3.75	4.24
W_3	(1.5)(14.0)(.15)	3.15		6.75	21.26
W_4	(4.17)(35)(.15)	21.89		25	547.31
W_5	(1.5)(14.0)(.15)	3.15		43.25	136.24
W_6	(7.5)(1.0)(.15)	1.13		46.25	52.26
W_7	½(14.0)(2.0)(.15)	2.10		44.67	93.80
S_1	(11.0)(4.0)(.14)	6.16		2.0	12.32
S_2	½(11.0)(1.57)(.14)	1.21		4.52	5.47
S_3	(2.5)(35)(.12)	10.5		25.0	262.50
S_4	½(12.0)(1.71)(.14)	1.44		45.43	65.42
S_5	(4.0)(12.0)(.14)	6.72		48.0	322.56
U_1	− 10(.0625)(50)	− 31.25		25.0	− 781.25
U_2	− ½(.0625)(1)(50)	− 1.56		33.33	− 52.08
Pe_1	½(.5)(.14)(2)²		.14	10.67	1.49
Pe_2	(.5)(.14)(2)(10)		1.4	5.0	7.00
Pe_3	½(.5)(.078)(10)²		1.95	3.3	6.44
Pe_4	− ½(.5)(.14)(2)²		− .14	11.67	− 1.63
Pe_5	− (.5)(.14)(2)(11)		− 1.54	5.5	− 8.47
Pe_6	− ½(.5)(.078)(11)²		− 2.36	3.67	− 8.66
Pw_1	½(.0625)(10)²		3.13	3.3	10.33
Pw_2	− ½(.0625)(11)²		− 3.78	3.67	− 13.87
Σ	—	ΣV = 27.87	ΣH = −1.20	—	ΣM = 693.87

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE / DOWNSTREAM
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Stability at Toe el. 354:

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{693.87}{27.87} = 24.9$ within mid $\frac{1}{3}$
  100% in bearing > 75% OK
- Sliding: $SF = \frac{\Sigma V \tan\phi + \cancel{c\ell}^{\,0}}{\Sigma H} = \frac{27.87(.53)}{1.2} \Rightarrow 12.31 > 1.33$ OK ✓
- Bearing: $f_{\pm} = \frac{\Sigma V}{area} \pm \frac{Mc}{I}$ $I = \frac{(1)(50)^3}{12} = 10{,}416.7$

$$f_{\pm} = \frac{27.82}{50} \pm \frac{27.82(0.1)(25)}{10{,}416.7} \Rightarrow .56 \pm .01 \Rightarrow f_+ = .57 \ ^{k}/_{ft^2}, \quad f_- = .55 \ ^{k}/_{ft^2} \Bigg\} < 4 \ ^{k}/_{ft^2} \ \text{OK}$$

Stability ⊥ to the stop log centerline:

Flood level ⇒ 366.0
Ground level behind stop logs. ⇒ 361 } 5 ft diffential head.

$P_W = \frac{1}{2}(.0625)(5)^2$ ⇒ 27.34 k

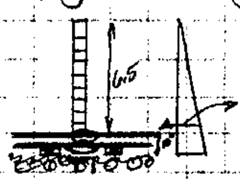
Weight of structural wedge (including uplift) ⇒ $27.87 \ ^{k}/_{ft_{strip}} \times 8$ ft wide
$\underline{222.96 \ K}$

Sliding safety factor ⇒ $\frac{\Sigma V \tan\phi + \cancel{c\ell}^{\,0}}{\Sigma H} \Rightarrow \frac{(222.96)(\tan 28)}{27.34} = 4.34 > 1.33$
OK ✓

Therefore, the U-channel structure satisfies criteria for overturning, sliding, and bearing pressure.

Stop-logs: The opening will be sand bagged around the rails and logs will start at el. 361.5 (top of rail). The 100 yr. event is 366.

Log wall height required ⇒ 366.0 − 361.5 ⇒ 4.5 feet + 2 ft freeboard.
$\underline{6.5}$ ft of logs.



Max Pressure at bottom log: $6.5\,\gamma_w \Rightarrow 406.25 \ ^{lb}/_{ft^2}$

say timber is 10"×10" ⇒ 9½" dressed ⇒ $406.25 \ ^{lb}/_{ft^2} \times \frac{9.5}{12} = \underline{321.61 \ ^{lb}/_{ft}}$

$$M_{max} = \frac{w\ell^2}{8} = \frac{321.61(17.5)^2}{8} = 12{,}311.8 \ lb \cdot ft$$

$S_{10\times10} = 142.896 \ in^3$ $\sigma = M/S = \frac{(12311.8)(12)}{142.896} \Rightarrow \underline{1033.91 \ ^{lb}/_{in^2}}$

Ave. allowable bending stress extreme fiber (Oak, white, red) ~ 1200 psi   OK ✓

$\boxed{\text{Use 2 bays of 9 - 10"×10" timber logs 17.5 feet long.}}$

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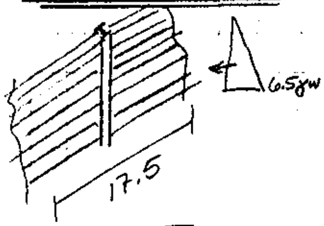
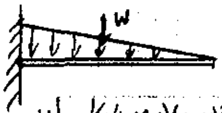
SUBJECT: Fort Fairfield — Maine LPP
COMPUTATION: Stop Log Structure / Downstream
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-6-87

Center Post: Max load $\Rightarrow (6.5)(17.5)(.0625\ kcf) = 7.11\ ^k/_{ft}$



Since the center post will be fixed at the base, it will act as a cantilever under a triangular load.



$M_{max} = \frac{Wl}{3} = \frac{(23.11)(6.5)}{3} \Rightarrow \underline{50.06\ k.ft}$

A36 steel
$f_b = .66 F_y = 23.76\ ksi$

$W = \frac{1}{2}(1.094)(6.5)^2$
$W = 23.11$

$V_{shear_{max}} = W = \underline{23.11\ k}$

$S_{req.} = \frac{M_{max}}{f_{bend.}} \Rightarrow \frac{50.06 \times 12''/'}{23.76} = 25.28\ in^3$ try W12x26 $S = 33.4\ in^3$
W12x22 $S = 25.4\ in^3$

Since the W-section must fit the 9½" stop log, a W12 section is needed.

$\boxed{\text{Use W12x26}}$ (larger flange.)

Deflection at free end $\delta = \frac{Wl^3}{15EI}$ $E = 29{,}000$ $I = 204\ in^4$

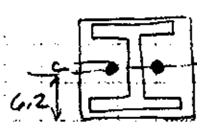
$\delta = \frac{(23.11)(6.5 \times 12)^3}{15(29000)(204)} \Rightarrow \underline{\delta_{max} = .123\ in \sim 1/8''}$

Brace not required however anchorage is important.

Anchorage:

① Anchor bolts: Assume A325 bolts. Shear at base = 23.11k

try 2 7/8"φ anchor bolts — $f_v = \frac{23.11}{2(.601)} = 19.26\ ksi$
Area = .601 in²



— $f_t = \frac{Mc}{I} \Rightarrow \frac{\overbrace{(23.11)(2.17 \times 12)}^{M}(.44)}{45.39} = 5.79\ ksi$

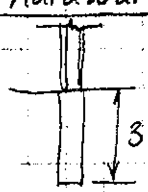
$I = \Sigma Ad^2$
$I = 2(.601)(6.2)^2$
$I = 45.39\ in^4$

— Allowables: $F_v = 21\ ksi$

for combined stress $F_t = 55 - 1.8 f_v \leq 44$
$F_t = 55 - 1.8(19.26)$
$= \underline{20.33} \leq 44$ $F_t = 20.33\ ksi$

$f_{t\ actual} = 5.79 < 20.33$ OK ✓
$f_{v\ actual} = 19.26 < 21$ OK ✓

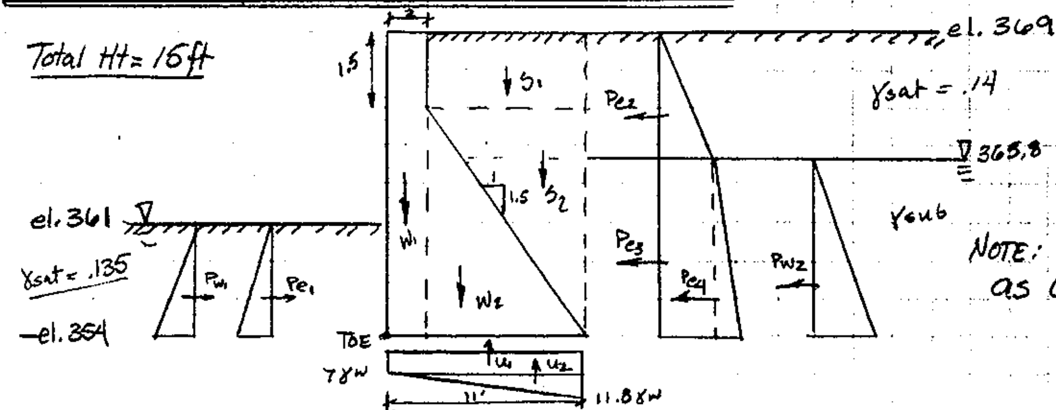
② No hardware: Fit W12x26 into a 7"x12" - 3 foot long depression



This should be adequate anchorage and the depression can be formed in the U-channel base slab.

SUBJECT FORT FAIRFIELD - MAINE LPP
COMPUTATION Gravity Walls / DOWNSTREAM
COMPUTED BY ENestorides CHECKED BY DATE 2-7-87

II GRAVITY WALL — SECTION A-A:



NOTE: Water level is determ... as 60% of unbalanced f...

Item	COMPUTATION	Vertical forces	Horizontal forces	Moment arm	MOMENT (+)
W_1	$2(15)(.15)$	4.5		1.0	4.5
W_2	$\frac{1}{2}(13.5)(9)(.15)$	9.11		5.0	45.55
S_1	$(1.5)(9)(.14)$	1.89		6.5	10.73
S_2	$\frac{1}{2}(13.5)(9)(.14)$	8.51		8.0	68.08
U_1	$-7(.0625)11$	-4.81		5.5	-26.46
U_2	$-\frac{1}{2}(.0625)(4.8)(11)$	-1.65		7.33	-12.1
P_{w1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{w2}	$-\frac{1}{2}(.0625)(11.8)^2$		-4.35	3.93	-17.11
P_{e1}	$\frac{1}{2}(.5)(.135)(7)^2$		1.65	2.33	3.85
P_{e2}	$-\frac{1}{2}(.5)(.14)(3.2)^2$		-.36	12.87	-4.63
P_{e3}	$-(.5)(.14)(3.2)(11.8)$		-2.64	5.9	-15.58
P_{e4}	$-\frac{1}{2}(.5)(.078)(11.8)^2$		-2.72	3.93	-10.70
Σ	—	ΣV = 17.55	ΣH = -6.89	—	ΣM = 49.7

Stability at Toe

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{49.7}{17.55} \Rightarrow 2.83$ % in bearing $= \frac{3(2.83)}{11} \times 100 \Rightarrow 77.2\% > 75$ OK

- Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\to 0}}{\Sigma H} = \frac{(17.55)\tan 28^\circ}{6.89} \Rightarrow 1.35 > 1.33$ OK

- Bearing Pressure: $f_{max} = \frac{2\Sigma V}{3a} \Rightarrow \frac{2(17.55)}{3(2.83)} \Rightarrow 4.13\ ^k/_{ft^2} \sim 4\ ^k/_{ft^2}$

Close enough for an extreme load condition.

OK

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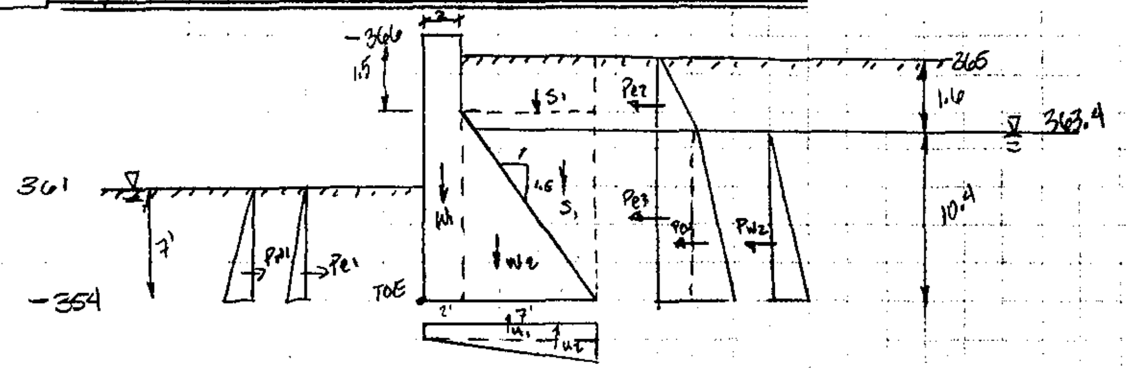
SUBJECT Fort Fairfield - Maine LPP

COMPUTATION Gravity Walls / Downstream

COMPUTED BY E Nestorides CHECKED BY ______ DATE 2-7-87

III. Gravity Wall - Section B-B:



Item	Computation	Vertical forces	Horizontal forces	Moment arm	Moment
W_1	$2(12)(.15)$	3.6		1.0	3.6
W_2	$\frac{1}{2}(10.5)(7)(.15)$	5.51		4.33	23.86
S_1	$1.6(7)(.14)$	1.57		5.5	8.64
S_2	$\frac{1}{2}(10.5)(7)(.14)$	5.15		6.67	34.35
U_1	$-7(9)(.0625)$	-3.94		4.5	-17.73
U_2	$-\frac{1}{2}(3.4)(9)(.0625)$	-.96		6.0	-5.76
P_{w1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{w2}	$-\frac{1}{2}(.0625)(10.4)^2$		-3.38	3.47	-11.73
P_{e1}	$\frac{1}{2}(.5)(.135)(7)^2$		1.65	2.33	3.85
P_{e2}	$-\frac{1}{2}(.5)(.14)(1.6)^2$		-.09	10.93	-.98
P_{e3}	$-(.5)(.14)(1.6)(10.4)$		-1.16	5.2	-6.03
P_{e4}	$-\frac{1}{2}(.5)(.078)(10.4)^2$		-2.11	3.47	-7.32
Σ	—	$\Sigma V = 10.93$	$\Sigma H = -3.56$	—	$\Sigma M = 28.32$

Stability at Toe

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{28.32}{10.93} = 2.59$ % in bearing $= \frac{3(2.59)}{9} \times 100\% = 86.4\% > 75$ OK ✓
- Sliding: $SF = \frac{\Sigma V \tan 28^\circ}{\Sigma H} = \frac{10.93(.53)}{3.56} \Rightarrow 1.63 > 1.33$ OK ✓
- Bearing: $f_{max} = \frac{2\Sigma V}{3a} \Rightarrow \frac{2(10.93)}{3(2.59)} \Rightarrow 2.81\ ^{k}/_{ft^2} < 4\ ^{k}/_{ft^2}$ OK ✓

SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: STOP-LOG STRUCTURE / DOWNSTREAM

COMPUTED BY: ENestorides CHECKED BY: DATE: 2-18-87

LOGS:

Change to 27' opening } 13.5' per log

From previous calculations: Max pressure at bottom Log ⇒ 406.25 lb/ft²

if timber is 8"x8" ⇒ 7½" dressed } $406.25 \times \frac{7.5}{12} = 253.91$ lb/ft

$M_{max} = \frac{wl^2}{8} = \frac{(253.6)(13.5)^2}{8} = 5784.3$ ft·lb

$S_{8\times8} = 70.313$ in³ $\sigma = M/S = \frac{(5784.3)(12''/1)}{70.313} = 987.18$ psi

allowable for white oak ~ 1200 psi OK.

Try 6"x6" ⇒ 5½ dressed $406.25 \times \frac{5.5}{12} \Rightarrow 186.20$ lb/ft

$M_{max} = \frac{(186.20)(13.5)^2}{8} = 4241.82$ ft.lb

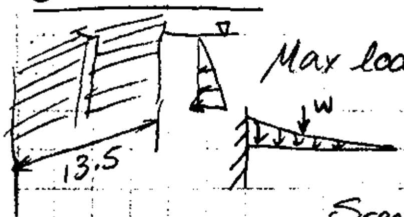
$S_{6\times6} = 27.729$ in³ $\sigma = M/S = \frac{4241.82 \times 12}{27.729} \Rightarrow 1835.69$ psi

NOT OK

USE 8" x 8" ⇒ 2 bays, each 13.5 ft long

Total: 22 logs required

Center Post:



Max load ⇒ $6.5(\gamma w)(13.5) \Rightarrow 5.48$ k/ft $W = \frac{1}{2}(\gamma w)(13.5)(6.5)^2$

$W = 17.82$

$M_{max} = \frac{Wl}{3} = \frac{17.82(6.5)}{3} = 38.62$ k.ft

$S_{req} = \frac{M_{max}}{f_{bend}} = \frac{(38.62 \times 12''/1)}{23.76} = 19.5$ in³

$f_b = 23.76$ ksi $= .66F_y$

Use W10x22 $S_x = 23.2$ in³

$b_f = 5.75$ in $k_1 = \frac{1}{2}$

Length of bearing $= b_f/2 - k_1 \Rightarrow 2.375$ in OK

STOP GROOVE:

8" groove

Center Post anchorage:

3-ft deep depression 6" x 11"

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SUBJECT: FORT FAIRFIELD - MAINE LPP.
COMPUTATION: STOP-LOG STRUCTURE - UPSTREAM
COMPUTED BY: ENestorides CHECKED BY: ____ DATE: 2/5/87

FORT FAIRFIELD: STOP LOG STRUCTURE / UPSTREAM

The stop-log structure is a U-channel with one bay of stop-logs. The southern wall will abut against an existing concrete retaining wall while the north wall will retain the dike embankment. The north wall will also extend past the channel section to include concrete retaining walls on either end.

The structures were designed in accordance to the following Corps Criteria:

1. EC 1110-2-510 "Working Draft of the Retaining and Flood wall Manual 31 Aug. 1983 w/ revisions 15 July 1985.
2. ETL 1110-2-256 "Sliding stability for Concrete structures" 24 June 1981.
3. EM 1110-2-2501 "Flood Wall Manual" Jan. 1948 w/ changes.

LENGTH OF CHANNEL:

The use of sheetpile cutoff is discouraged due to the hardness of the foundation material. Therefore, a U-channel length was chosen [based on the differental head occuring during a flood] to minimize the possibility of seepage under the channel.

EM 1110-2-2501 Flood Wall Manual Para. 108(c)

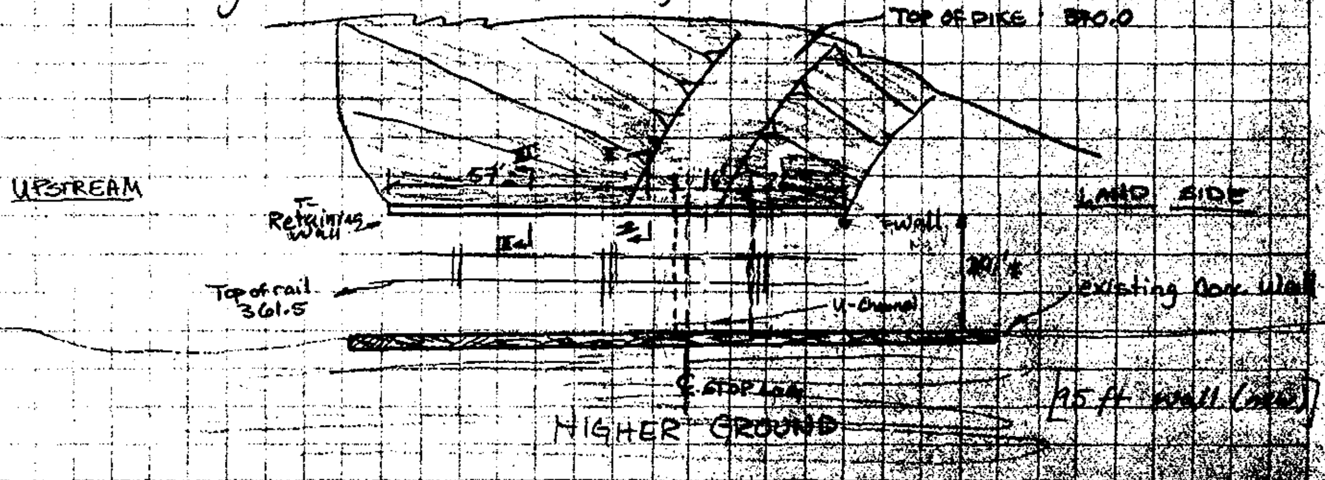
Creep Ratio $\Rightarrow \frac{\text{Line of Creep}}{\Delta \text{Head}}$ if ratio ≥ 4 (for sands, gravels no additional cutoff needed)

Δ Head $\Rightarrow$ (flood level) − (tail-ground level) $\Rightarrow$ (367.5 − 360.0) $\Rightarrow$ 7.5 ft

4×7.5 = min. line of creep $\Rightarrow$ 30.0 Use a 16.0 ft long section.

Frost depth 6 feet min.

The stop log structure and retaining walls are shown in plan below:



Each structure will be analysed separately in the pages to follow.

DESIGN METHOD:

One section of the concrete U-channel and two typical sections of the retaining wall will be analyzed for stability under one Load case: LOAD case R2 as described in the draft of the Retaining and Flood Wall Manual.

This load case represents a rapid drawdown situation which is an extreme loading for retaining structure. Since the area in question is in seismic zone One (minor damage) the earthquake condition is not critical.

All the structures are founded on compacted gravel fill or in certain areas, on the naturally deposited soils. Under Load Case R2, the structures must satisfy the following stability criteria:

a) 75% of the base must be in compression,
b) the factor of safety against sliding must be greater than 1.33,
c) bearing pressures should not exceed the allowable $4^{k}/_{ft^2}$.

Design and Soil Parameters:

	γ_{moist}	$\gamma_{saturated}$	$\gamma_{submerged}$	ϕ	c/δ
Dumped Gravel bedding (RR)	.120	.135	.073	32°	0/0°
Compacted Impervious	.135	.140	.078	30°	0/0°
foundation soil.	.130	.137	.075	28°	0/0°

- Consider Soil pressures using K_o ⇒ at rest coefficient.

$K_o = 1 - \sin\phi \quad \phi = 30° \qquad \boxed{K_o = .5}$

- For Sliding: $\mu = \tan\phi \quad \phi = 28°$, $c = 0$ tsf $\qquad \boxed{\mu = .53}$

- RAPID DRAWDOWN: This condition was modeled by treating 60% of the unbalanced fill as submerged and the remaining 40% as saturated.

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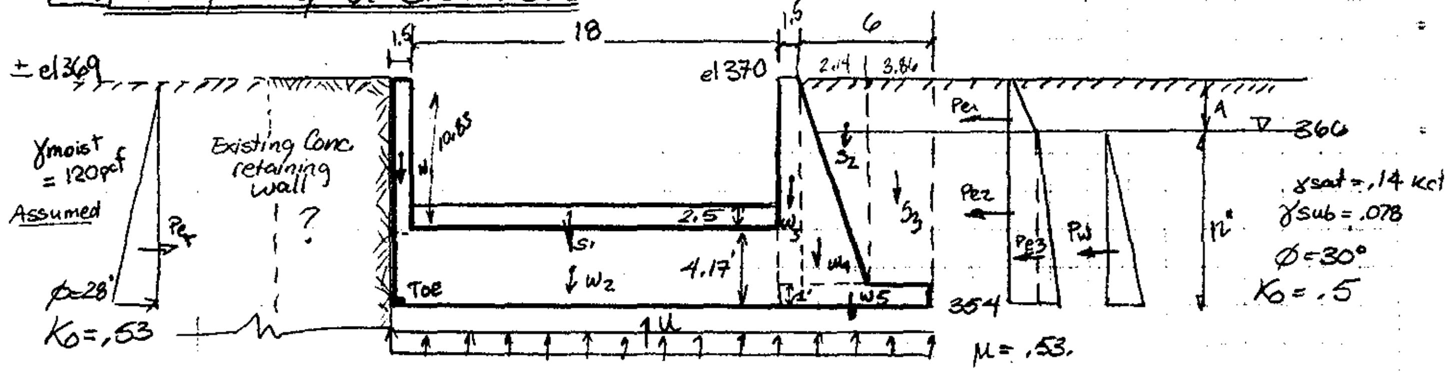
SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-9-87

I. Stop Log U-Channel:



The water line behind the battered U-channel wall is at el. 366. This elevation was used to roughly represent 60% of the unbalanced fill on both sides of the wall. The contribution of the existing concrete wall on the U-channel will be modeled as the height of light fill it could retain.

Item	Computation	Vertical Forces (K)	Horizontal forces (K)	Moment arm	Moment (K·Ft)
W_1	(1.5)(10.83)(.15)	2.44		.75	1.83
W_2	(4.17)(19.5)(.15)	12.20		9.75	118.92
W_3	(1.5)(15)(.15)	3.38		20.25	68.45
W_4	½(15)(2.14)(.15)	2.41		21.71	52.32
W_5	(1)(7.5)(.15)	1.05		23.25	24.41
U	-12(.0625)(27)	-20.25		13.50	-273.38
S_1	2.5(18)(.12)	5.40		10.50	56.70
S_2	½(15)(2.14)(.14)	2.25		22.43	50.47
S_3	(3.86)(15)(.14)	8.11		25.07	203.32
P_{er}	½(.53)(.12)(15)²		7.16	5.0	35.80
P_{e1}	-½(.5)(.14)(4)²		-.56	13.33	-7.47
P_{e2}	-(.5)(.14)(4)(12)		-3.36	6.0	-20.16
P_{e3}	-½(.5)(.078)(12)²		-2.81	4.0	-11.24
P_W	-½(.0625)(12)²		-4.50	4.0	-18.00
Σ	—	ΣV= 16.99	ΣH= -4.07	—	ΣM= 281.97

Stability at toe:

- Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{281.97}{16.99} = 16.60$ → within kern 100% > 75% in bearing OK

- Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\circ}}{\Sigma H} \Rightarrow \frac{16.99(.53)}{4.07} \Rightarrow 2.21 > 1.33$ OK

- Bearing Pressure: $f_{\pm} = \frac{16.99}{27.0} \pm \frac{(16.99)(-3.1)(13.5)}{1640.25}$ $I = 1640.25$

$f_{\pm} = .63 \pm (-.43) \Rightarrow$ $f_+ = .20\ K/ft^2$, $f_- = 1.06\ K/ft^2$ < 4K/ft² OK

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SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: STOP LOG STRUCTURE / UPSTREAM
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-9-87

StopLogs:

Topof rail ⇒ 361.5 3675-361.5 ⇒ 6.0 ft. + 2 freeboard.
Design flood ⇒ 367.5 ⇒ 8.0


7.5 Max. Pressure ⇒ 8.0(.0625) ⇒ .50 k/ft²

Try 10"x10" ~ 9½" dressed .50 × $\frac{9.5}{12}$ ⇒ .395 k/ft

Opening is 18' wide ⇒ max. Moment = $\frac{wl^2}{8}$ = $\frac{(.395)(18)^2}{8}$ ⇒ 16.03 k.ft.

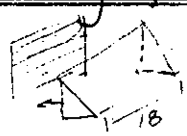
10" 10" S_{10x10} = 142.896 in³ σ = M/S = $\frac{16.03 \times 12''/'}{142.896}$ ⇒ σ = 1.35 k/in²

12" 8" S_{8x12} = 165.313 .5 × $\frac{7.5}{12}$ = .313 M = $\frac{(.313)(18)^2}{8}$ = 12.66 k.ft.

σ = $\frac{12.66(12)}{165.313}$ ⇒ .92 k/in²

σ = allowable (white, oak) 1200 psi ⟶ use 8"x12" section
(12 required.)

Sliding of Structure due to Hydrostatic pressure: flood 367.5
topofrail ⇒ 360.0 } ΔH=7.5


7.5 ½ .0625(7.5)²× 18 ⇒ 31.64 k

Weightofstructure ⇒ 9.08 k/ft ×16 ft ⇒ 145.28
including uplift.

Sliding ⇒ $\frac{145.28(.53)}{31.64}$ ⇒ 2.43 > 1.33 OK ✓

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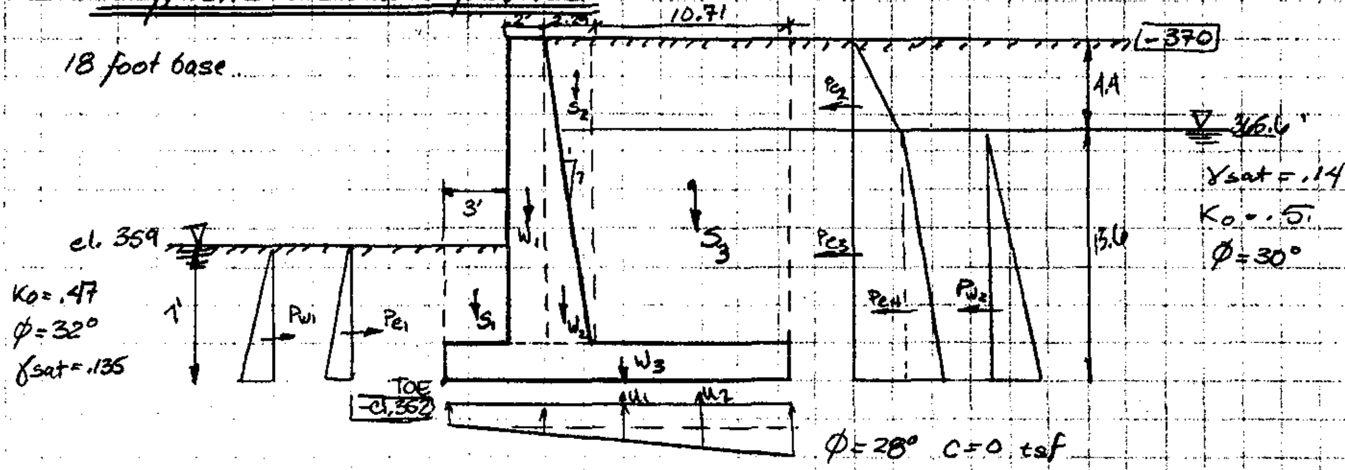
SUBJECT Fort Fairfield Maine - LPP
COMPUTATION StopLog Retaining Wall / Upstream
COMPUTED BY ENestorides CHECKED BY DATE 2/10/87

II. Typical Retaining Wall:



Item	COMPUTATIONS	Vertical forces	Horizontal forces	Moment arm	Moment
W_1	$2(16)(.15)$	4.8		4.0	19.2
W_2	$\frac{1}{2}(16)(2.29)(.15)$	2.75		5.76	15.84
W_3	$2(18)(.15)$	5.4		9.0	48.6
S_1	$3(5)(.135)$	2.03		1.5	3.05
S_2	$\frac{1}{2}(16)(2.29)(.14)$	2.56		6.53	16.71
S_3	$10.71(16)(.14)$	23.99		12.65	303.47
U_1	$-\ 7(.0625)18$	−7.88		9.0	−70.88
U_2	$-\ \frac{1}{2}(.0625)(6.6)(18)$	−3.71		12.0	−44.56
Pe_1	$\frac{1}{2}(.47)(.135)(7)^2$		.86	2.33	2.01
Pe_2	$-\ \frac{1}{2}(.5)(.14)(4.4)^2$		−.68	15.07	−10.21
Pe_3	$-\ (.5)(.14)(4.4)(13.6)$		−4.19	6.8	−28.48
Pe_4	$-\ \frac{1}{2}(.5)(.078)(13.6)^2$		−3.61	4.53	−16.34
Pw_1	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
Pw_2	$-\ \frac{1}{2}(.0625)(13.6)^2$		−5.78	4.53	−26.18
Σ		$\Sigma V = 29.94$	$\Sigma H = -11.87$		$\Sigma M = 215.81$

Stability at toe -352

-Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{215.81}{29.94} \Rightarrow 7.2$ in mid $\frac{1}{3} \Rightarrow$ 100% in bearing > 75% OK

-Sliding: $SF = \frac{\Sigma V \tan\phi + cL^{\to 0}}{\Sigma H} = 1.34 > 1.33$ OK

-Bearing Pressure: $f_\pm = \frac{\Sigma V}{area} \pm \frac{Mc}{I}$ $I = \frac{1(18)^3}{12} \Rightarrow 486\ ft^4$

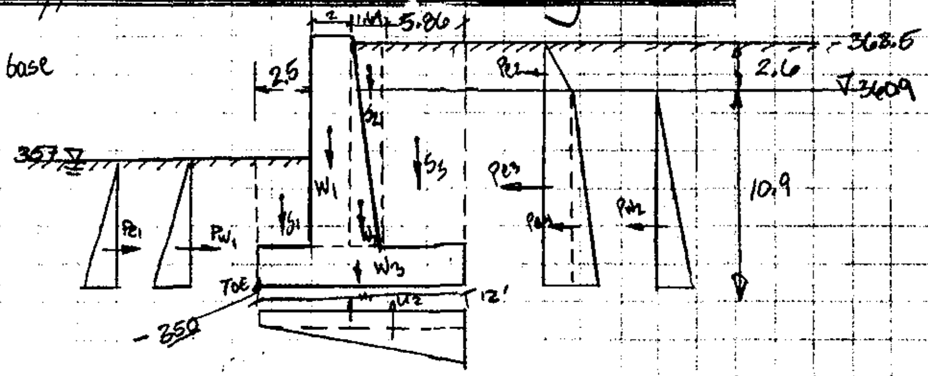
$f_\pm = \frac{29.94}{18} \pm \frac{29.94(1.8)(9)}{486} \Rightarrow 1.66 \pm .998$

$f_+ = 2.66\ k/ft^2$
$f_- = 0.66\ k/ft^2$ $\Big\} < 4\ k/ft^2$ OK

SUBJECT Fort Fairfield – Maine LPP
COMPUTATION Stop Log Retaining Wall / Upstream
COMPUTED BY E Nestorides CHECKED BY DATE 2-10-87

III. Typical Section III: Retaining wall

12 ft base



Item	Computations	Vertical forces	Horizontal forces	Moment arm	Moment (+)
W_1	$2(11.5)(.15)$	3.45		3.5	12.08
W_2	$\frac{1}{2}(11.5)(1.64)(.15)$	1.41		5.05	7.12
W_3	$2(12)(.15)$	3.6		6.0	21.6
S_1	$(2.5)(5)(.135)$	1.69		1.25	2.11
S_2	$\frac{1}{2}(11.5)(1.64)(.14)$	1.32		5.59	7.38
S_3	$(5.86)(11.5)(.14)$	9.43		9.07	85.53
U_1	$-7(12)(.0625)$	−5.25		6.0	−31.5
U_2	$-\frac{1}{2}(3.9)(12)(.0625)$	−1.46		8.0	−11.68
P_{w1}	$\frac{1}{2}(.0625)(7)^2$		1.53	2.33	3.57
P_{w2}	$-\frac{1}{2}(.0625)(10.9)^2$		−3.71	3.63	−13.47
P_{e1}	$\frac{1}{2}(.47)(.075)(7)^2$		.86	2.33	2.00
P_{e2}	$-\frac{1}{2}(.5)(.14)(2.6)^2$		−.24	11.77	−2.82
P_{e3}	$-(.5)(.14)(2.6)(10.9)$		−1.98	5.45	−10.79
P_{e4}	$-\frac{1}{2}(.5)(.078)(10.9)^2$		−2.32	3.63	−8.42
Σ	—	$\Sigma V = 14.18$	$\Sigma H = 5.86$	—	$\Sigma M = 62.71$

Stability at Toe el. 360.0

– Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{62.71}{14.18} = 4.42$ in mid. ⅓ ⇒ 100% bearing > 75% OK

– Sliding: $SF = \frac{14.18 \tan 28^\circ}{5.86} \Rightarrow 1.29 < 1.33$ close enough. OK

– Base Pressures: $f_\pm = \frac{14.18}{12} \pm \frac{(14.18)(1.68)(6)}{144} = 1.18 \pm .93$

$$\left.\begin{array}{l} f_+ = 2.11 \ ^k/_{ft^2} \\ f_- = .25 \ ^k/_{ft^2} \end{array}\right\} < 4 \ ^k/_{ft^2} \quad \text{OK}$$

SUBJECT Fort Fairfield – Maine
COMPUTATION Drainage pipe head-wall
COMPUTED BY S. Gagnon CHECKED BY DATE 2-13-87

DESIGN Method:

The headwall is designed and analyzed for stability as one unit. Criteria evolved from the Hydraulics appendix included:

1. Head wall soil elevation = 371
2. Side wall soil elevation ≈ 370.0 (estimate from topography map)
3. Pipe invert elevation ≈ 363.7 Min. (4' Ø Pipe)

The load case used represents a rapid drawdown situation which is an extreme loading for a retaining structure of this sort. Since the seismic zone is one for this area, the earthquake condition is not critical.

The structure is designed for a foundation of compacted gravel fill. Under the rapid draw back load case, the structure must satisfy the following stability criteria:

1. 75% of the base must be in compression.
2. Factor of Safety against sliding must exceed 1.33.
3. Bearing pressures should not exceed the allowable 4 K/ft²

Design and Soil Parameters:

	γ_{sat}	γ_{sub}	ϕ	c
Compacted gravel Fill	.145	.0825	32°	0

- Soil Pressures using Ko ⇒ at rest coefficient.

$K_o = 1 - \sin\phi \quad \phi = 32° \quad K_o = .47$

- Sliding: $\mu = \tan\phi \quad \phi = 32° \quad \mu = .62$
  $c = 0$

- Rapid Drawdown: This condition was modeled by using 40% of the unbalanced fill as submerged and the remaining as saturated. The 40% is representable as a drainable fill.

SUBJECT Fort Fairfield - Maine
COMPUTATION Drainage pipe headwall
COMPUTED BY S. Guynn CHECKED BY DATE 2-13-87

The headwall is designed as both a retaining and outlet structure.
The structure is designed in accordance to the following Corps criteria:

1. EC 1110-2-510 "Working Draft of the Retaining and Flood wall Manual" 31 Aug. 1983 w/revisions 15 July 1985
2. ETL 1110-2-256 "sliding stability for Concrete structures." 24 June 1981.

Soil and water pressures are developed from the following equations:

$$\text{Hydr. pressure} = \gamma_w h$$

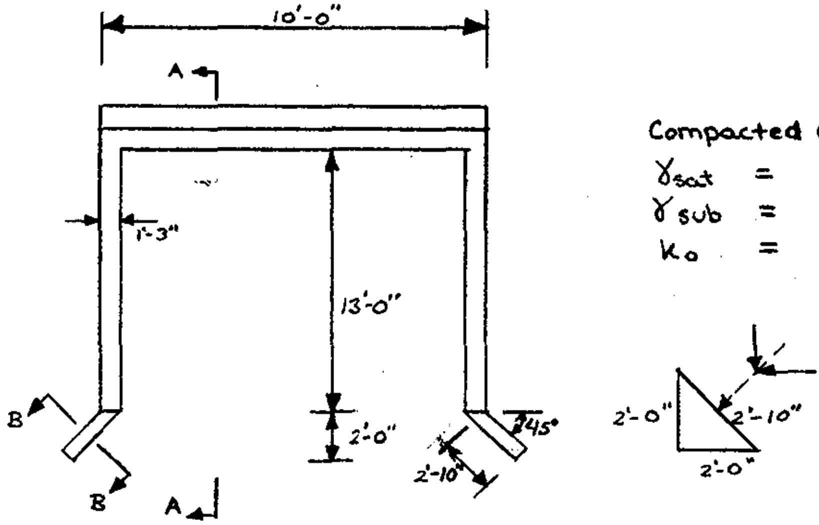
$$\text{Saturated Soil Pressure} = \gamma_{sat}(h)(K_o)$$

$$\text{Submerged Soil Pressure} = \gamma_{sub}(h)(K_o)$$

SUBJECT

COMPUTATION

COMPUTED BY CHECKED BY DATE

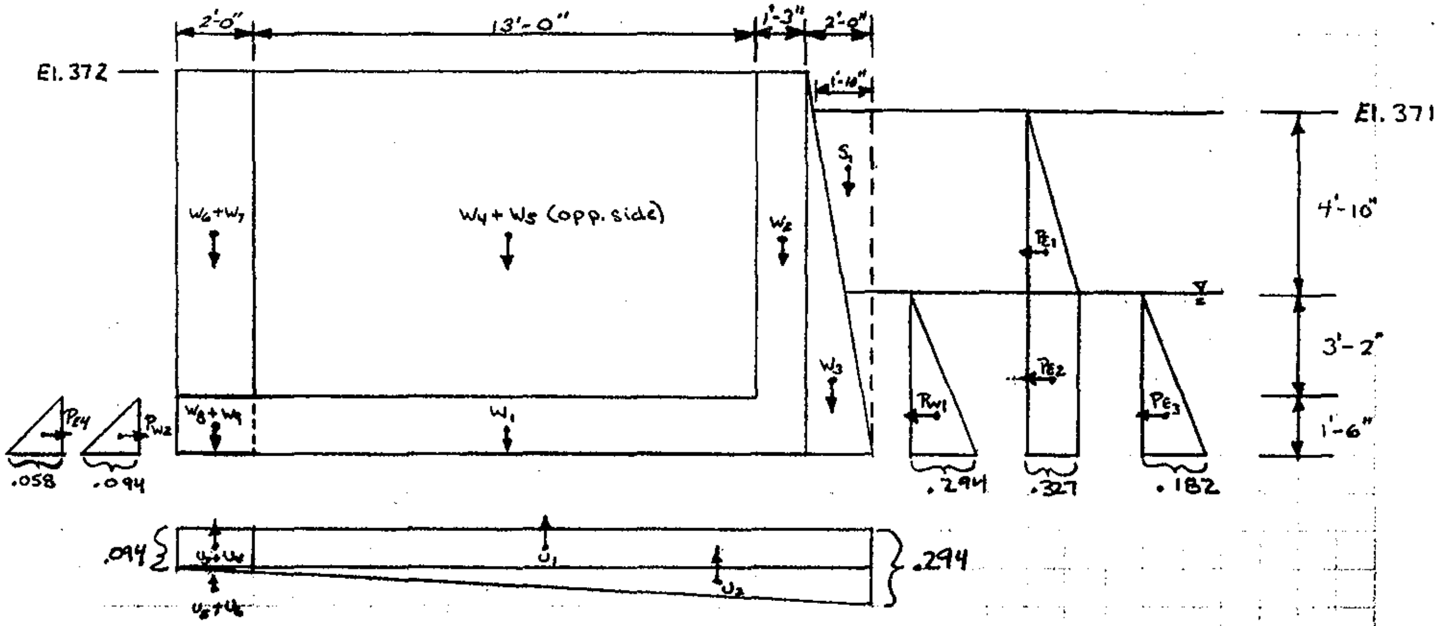


Compacted Gravel

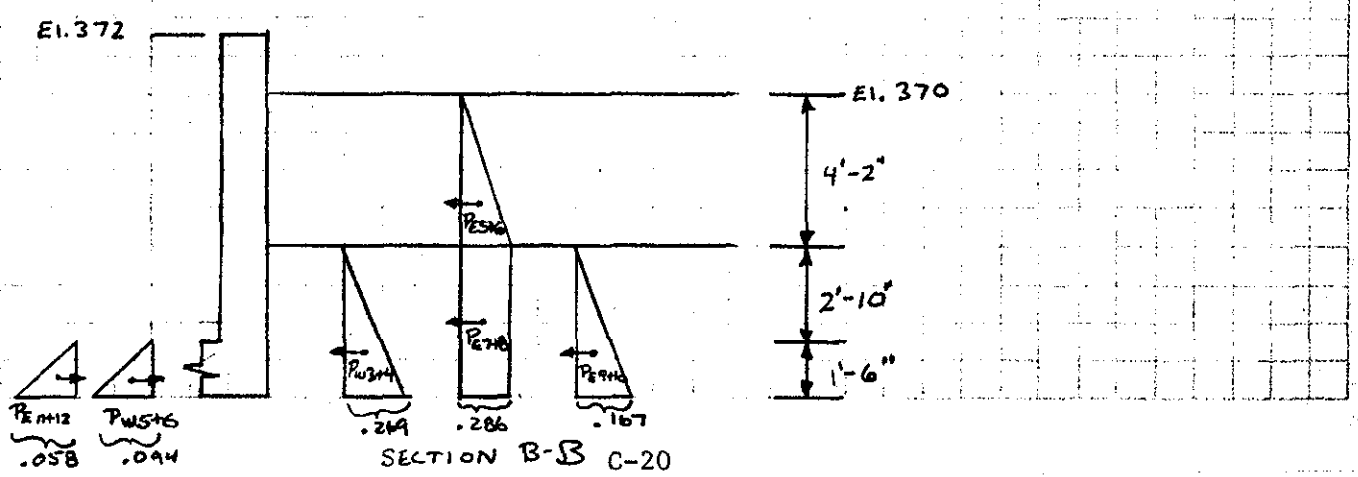
$\gamma_{sat} = 145$ lb/ft^3

$\gamma_{sub} = 82.6$ lb/ft^3

$k_o = 0.47$



SECTION A-A



SECTION B-B

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SUBJECT Fort Fairfield - Maine

COMPUTATION Drain-pip headwall

COMPUTED BY [signature] CHECKED BY DATE

ITEM	Computation	Vertical Forces (K)	Horizontal Forces (K)	Moment Arm (ft)	↶ Moment @ TOE (K-ft)
W_1	(1.5)(15)(10)(.15)	+33.75		7.5	+253.12
W_2	(1.25)(10.5)(10)(.15)	+19.68		15.625	+307.5
W_3	1/2(2)(10.5)(10)(.15)	+15.75		16.92	+266.5
W_4+W_5	2(1.25)(13)(9)(.15)	+43.87		8.5	+372.89
W_5+W_6	2(2.83)(1.25)(9)(.15)	+9.55		1.0	+9.55
W_7+W_8	2(1/2)(2)(2)(1.5)(.15)	+0.9		0.667	+0.60
S_1	1/2(1.81)(9.5)(10)(.145)	+12.47		17.67	+220.34
U_1	(.094)(10)(18.25)	−17.15		9.125	−156.53
U_2	(1/2)(.2)(10)(18.25)	−18.25		12.167	−220.05
U_3+U_4	1/2(2)(.094)(2)(2)	−.376		1.0	−.376
U_5+U_6	1/2(.175)(2)(2)	−.350		1.33	−.465
P_{W1}	1/2(.294)(4.7)(10)		−6.9	1.567	−10.81
P_{W2}	1/2(.094)(1.5)(10)		+0.705	0.5	+0.352
$P_{W3}+P_{W4}$	2(1/2)(.269)(4.3)(2)		−2.31	1.43	−3.30
$P_{W5}+P_{W6}$	2(1/2)(.094)(1.5)(2)		+.282	0.5	+.141
P_{E1}	1/2(.327)(4.8)(10)		−7.85	6.3	−49.44
P_{E2}	(.327)(4.7)(10)		−15.37	2.35	−36.12
P_{E3}	(1/2)(.182)(4.7)(10)		−4.28	1.567	−6.70
P_{E4}	(1/2)(.058)(1.5)(10)		+.435	0.5	+.217
$P_{E5}+P_{E6}$	2(1/2)(.286)(4.2)(2)		−2.40	5.7	−13.69
$P_{E7}+P_{E8}$	2(4.3)(.286)(2)		−4.92	2.15	−10.57
$P_{E9}+P_{E10}$	2(1/2)(.167)(4.3)(2)		−1.44	1.43	−2.05
$P_{E11}+P_{E12}$	2(1/2)(.058)(1.5)(2)		+.174	0.5	+.087

$\Sigma V = 99.84$ $\Sigma H = 43.87$ $\Sigma M_{@TOE} = 921.98$

Stability @ TOE

− overturning: $\frac{\Sigma M}{\Sigma V} = \frac{921.98}{99.84} = 9.23 > \frac{18.25}{3} < 12.167$ ok

− Sliding: $SF = \frac{\Sigma V \tan\phi + c\cancel{L}^{0}}{\Sigma H} = \frac{(99.84)(.62)}{43.87} = 1.41 > 1.33$ ok

− Bearing Pressure: $f = \pm \frac{99.84}{154.0} \pm \frac{99.84(.105)(9.125)}{5065}$

(I = 5065)

$f\pm = .65 \pm .019 \Rightarrow \left.\begin{array}{l} f+ = .669 \\ f- = .631 \end{array}\right\} < 4.0$ K/ft² ok

FORT FAIRFIELD - PUMP STATION

The pumping station was checked for stability along the bottom foundation depth of 346.3, in the north/south direction.

The structure was analyzed in accordance to the following Corps Criteria:

1. EC 1110-2-510: "Working draft of the Retaining and Flood Wall Manual" 31 August 1983 (w/Changes 15 July 85)

2. ETL 1110-2-256: "Sliding Stability of concrete structures." 24 June 1981.

The soil surrounding the pumping station is at el. 360. The "worst" case loading involved the soil saturated all around the pumping station. Since Fort Fairfield is in seismic zone One (minor damage), the earthquake load is not the critical loading.

The criteria for stability that must be satisfied are as follows:

a) That, the factor of safety against sliding is greater than 1.5 (this is an assumed value found acceptable for the given building and load case)

b) that the bearing pressures do not exceed the allowable $4\,k/ft^2$,

c) and, that 100% of the base be in compression.

Attached are sections upon which the weight of the structure was calculated.

Soil Parameters: Ko - at rest will be used for lateral soil pressures. The effects of the soil in the east/west direction resisting the overturning moment were not considered to be more conservative.

Assume: Compacted gravel fill around pumping station: $\gamma_{moist} = 135$, $\gamma_{sat} = 145$, $\phi = 35°$

$$K_o = 1 - \sin\phi = .43$$

Foundation Soil $\rightsquigarrow$ GEB $\Rightarrow$ $\phi = 28°$, $C = 0$ tsf, $\mu = \tan\phi = .53$

allowable bearing capacity $= 4^{K}/ft^2$

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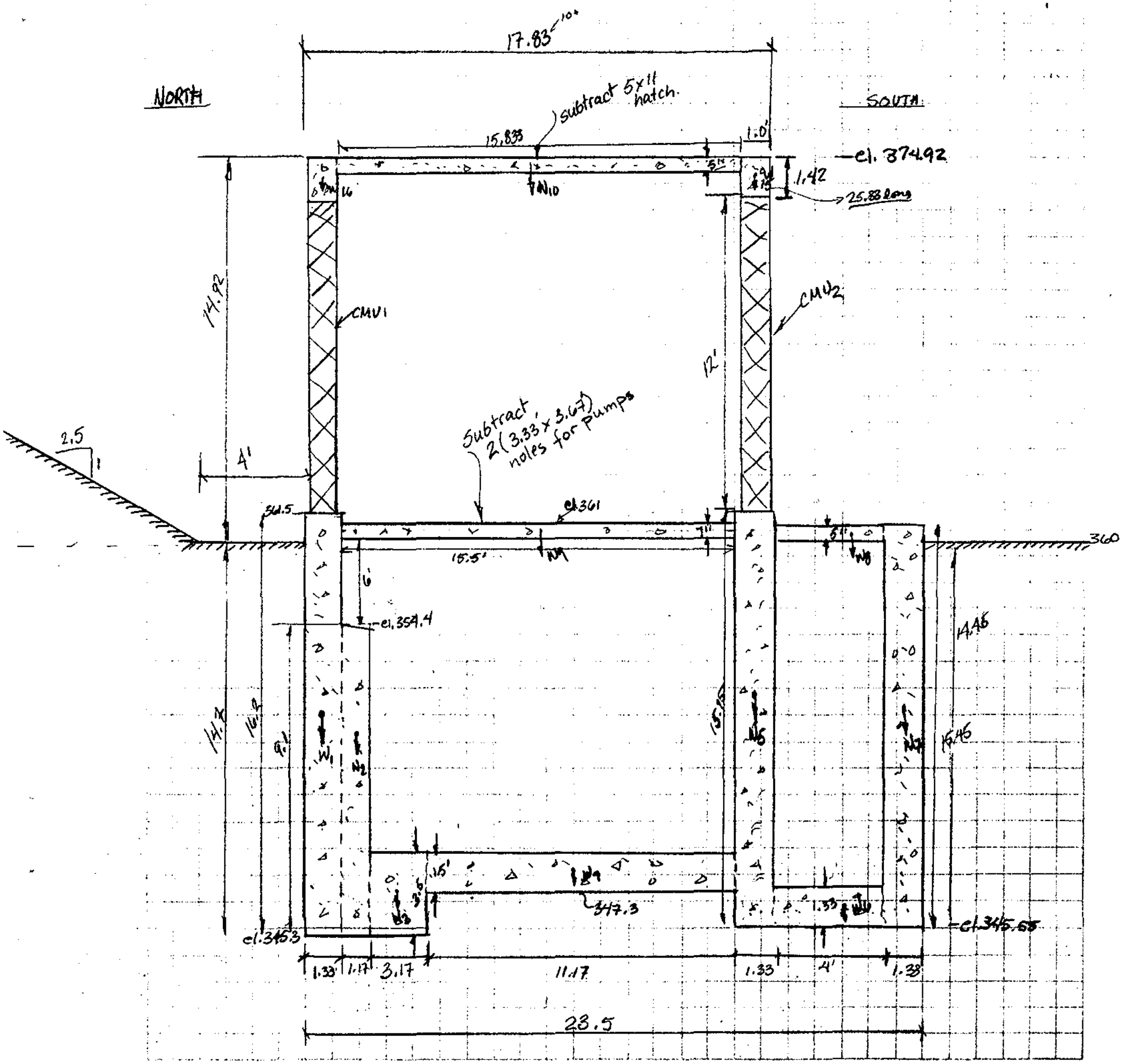
SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: PUMPING STATION STABILITY

COMPUTED BY: ENestorides CHECKED BY: ______ DATE: 2-12-87

PUMPING STATION TYPICAL SECTION.

ELEVATION NORTH / SOUTH



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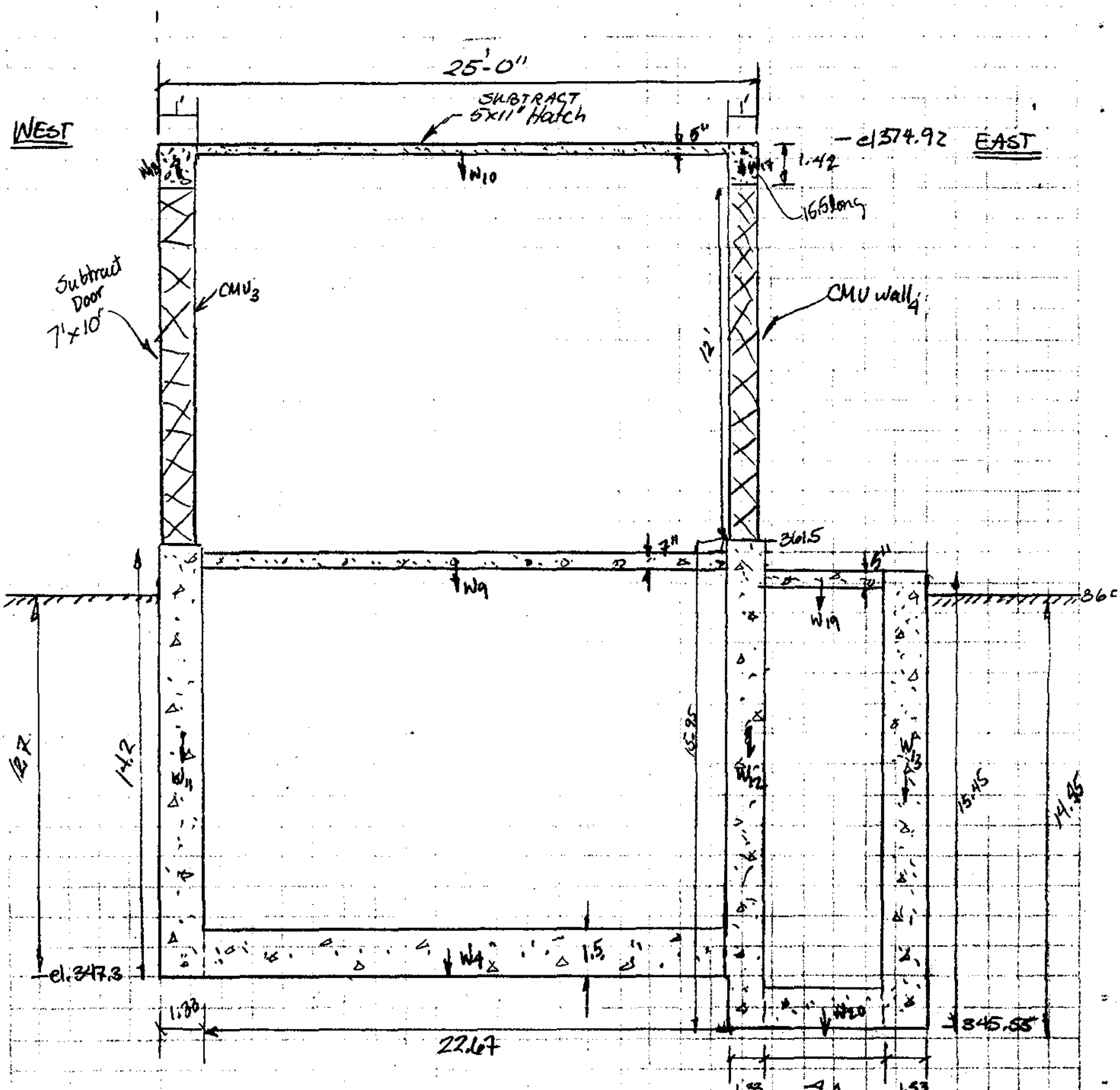
SUBJECT FORT FAIRFIELD - MAINE LPP

COMPUTATION PUMPING STATION STABILITY

COMPUTED BY ENestorides CHECKED BY DATE 2-12-87



ELEVATION
WEST / EAST

NOT TO ACCURATE SCALE

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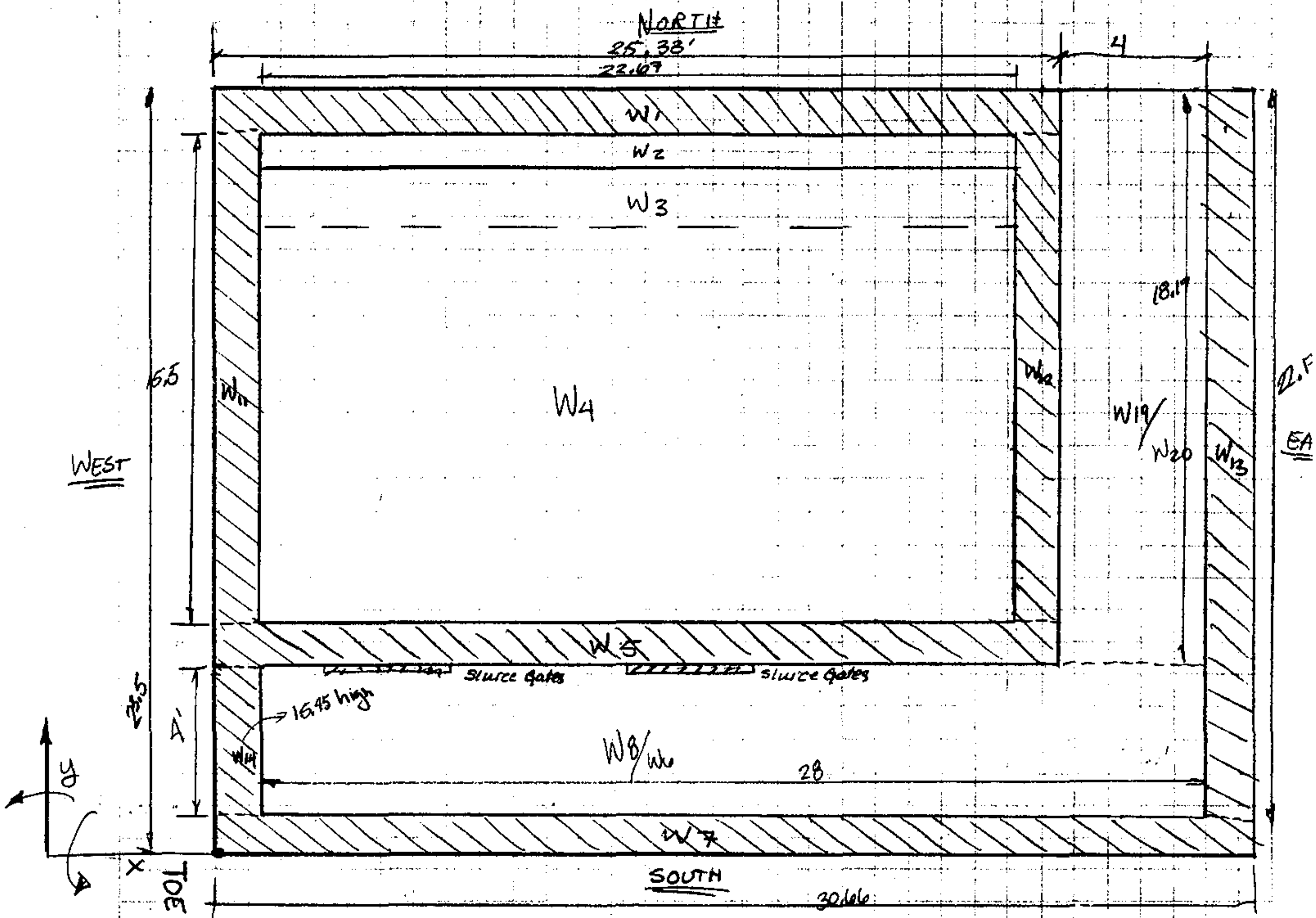
SUBJECT Fort Fairfield - Maine

COMPUTATION Pumping Station Stability

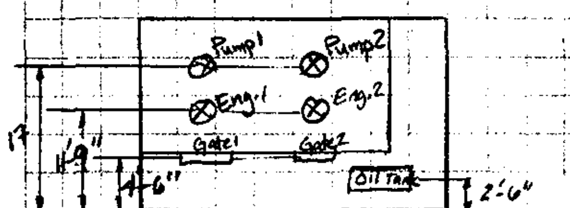
COMPUTED BY ENestorides CHECKED BY DATE 2/12-87



Foundation Plan

Not to Accurate Scale

EQUIPMENT LAYOUT:



Pump ⇒ 3.0 k each
Engine ⇒ 3.3 k each
Gates ⇒ 1.3 k each (w/motor)
Oil Tank ⇒ 4 k

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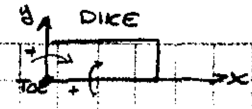
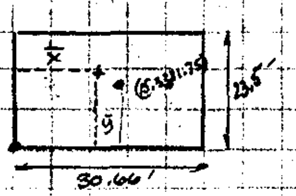
SUBJECT Fort Fairfield – Maine LPP
COMPUTATION Pumping Station Stability
COMPUTED BY ENestorides CHECKED BY DATE 2-12-87



WEIGHT OF STRUCTURE AND CENTER OF GRAVITY

Item	COMPUTATION	WEIGHT (K)	$\bar{y}$ Moment arm	Mx Moment about x-axis	$\bar{x}$ moment arm	My Moment (k-f) arm
Concrete						
W1	(1.33)(16.2)(25.33)(.15)	81.86	22.84	1869.77	12.67	1037.22
W2	(1.17)(9.1)(22.67)(.15)	36.21	21.59	781.67	12.67	458.72
W3	(3.17)(3.5)(22.67)(.15)	37.73	19.42	732.69	12.67	478.02
W4	(11.17)(1.5)(22.67)(.15)	56.98	12.25	697.95	12.67	721.94
W5	(1.33)(15.95)(25.33)(.15)	80.60	6.0	483.60	12.67	1021.21
W6	(4.0)(1.33)(28)(.15)	22.34	3.33	74.41	15.33	342.53
W7	(1.33)(15.45)(30.66)(.15)	94.50	.67	63.32	15.33	1448.72
W8	(4.0)(5/12)(28)(.15)	7.06	3.33	23.50	15.33	108.17
W9	(15.5)(7/12)(22.67)(.15) − 2(3.33)(3.67)(.15)(.58)	28.44	14.42	410.16	12.67	360.39
W10	(15.83)(.42)(23.0)(.15) − (5)(11)(.15)(.42)	19.47	14.59	284.11	12.50	243.41
W11	(1.33)(14.2)(15.5)(.15)	43.91	14.42	633.18	.67	29.42
W12	(1.33)(15.95)(15.5)(.15)	49.32	14.42	711.21	24.66	1216.27
W13	(1.33)(15.45)(22.17)(.15)	68.33	12.42	848.71	29.99	2049.34
W14	(4.0)(15.45)(1.33)(.15)	12.33	3.33	41.06	.67	8.26
W15	(1.0)(1.42)(25.33)(.15)	5.40	6.17	33.29	12.67	68.36
W16	(1.0)(1.42)(25.33)(.15)	5.40	23.0	124.09	12.67	68.36
W17	(1.42)(1.0)(15.5)(.15)	3.30	14.42	47.61	24.33	80.33
W18	(1.42)(1.0)(15.5)(.15)	3.30	14.42	47.61	.67	2.21
W19	(4.0)(.142)(18.19)(.15)	4.58	14.42	66.10	27.33	125.28
W20	(4.0)(1.33)(18.19)(.15)	14.52	14.42	209.32	27.33	396.91
CMU Walls						
CMU1	(12)(1)(25.33)(.08)	24.32	22.84	555.40	12.67	308.09
CMU2	(12)(1)(25.33)(.08)	24.32	5.84	142.01	12.67	308.09
CMU3	[(12)(1)(15.5) − (1)(7)(10)](.08)	9.28	14.42	133.82	.67	6.22
CMU4	(12)(1)(15.5)(.08)	14.88	14.42	214.57	24.66	366.94
Σ		Ws = 748.38	—	ΣMx = 9,229.16	—	ΣMy = 11,254.21



$$\bar{x} = \frac{\Sigma M_y}{W_s} = \frac{11254.21}{748.38} \qquad \bar{x} = 15.04 \text{ ft}$$

$$\bar{y} = \frac{\Sigma M_x}{W_s} = \frac{9229.16}{748.38} \qquad \bar{y} = 12.33 \text{ ft}$$

$$I_x \Rightarrow \frac{(30.66)(23.5)^3}{12} = 33158 \text{ ft}^4 \qquad \text{Area} \Rightarrow (23.5)(30.66) \Rightarrow 720.51$$

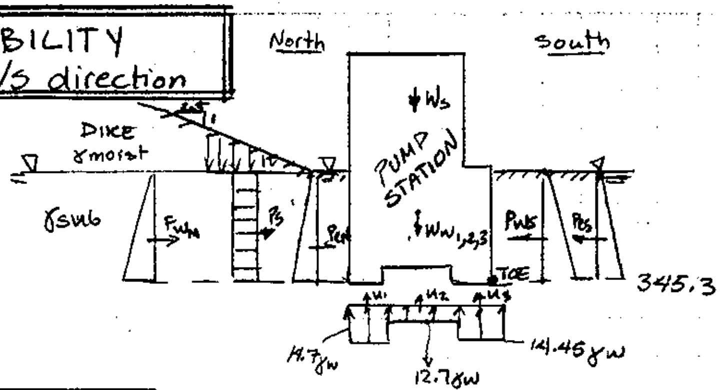
$$I_y = \frac{(23.5)(30.66)^3}{12} = 56442 \text{ ft}^4$$

SUBJECT: FORT FAIRFIELD - MAINE LPP

COMPUTATION: PUMPING STATION STABILITY

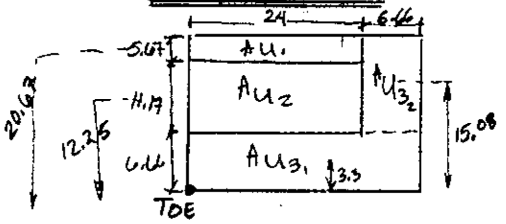
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-12-87

STABILITY
N/S direction



LOAD CASE 1: Flood Conditions, surrounding backfill submerged, water in sump chamber.

1. UPLIFT:



Uplift felt on base slab.

$A_{U_1} \Rightarrow 136.08$ $\uparrow U_1 = (136.08)[14.7(.0625)] = \underline{125.02\ k}$

$A_{U_2} \Rightarrow 268.08$ $\uparrow U_2 = (268.08)[(12.7)(.0625)] = \underline{212.79\ k}$

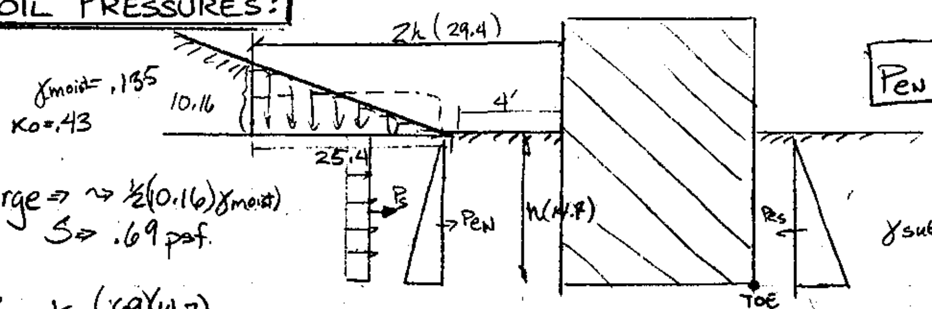
$A_{U_3} \Rightarrow 316.35$ $\uparrow U_3 \Rightarrow (316.35)[(14.45)(.0625)] = 285.79\ k$

$\uparrow U_{3_1} = \underline{184.42\ k}$

$\uparrow U_{3_2} = \underline{101.29\ k}$

- HORIZONTAL HYDROSTATIC FORCES CANCEL OUT.

2. SOIL PRESSURES:



$P_{EN} = P_{ES}$ and Cancel out

Surcharge $\Rightarrow \sim \frac{1}{2}(0.16)\gamma_{moist}$
$S \Rightarrow .69$ psf.

$P_S = K_o(.69)(14.7)$
$\Rightarrow 4.33$ k/f linear.

Resultant $P_S \Rightarrow 4.33$ k/ft $\times 30.66 = \underline{132.91\ k}$ at 7.35 ft above toe

3. WATER IN SUMP CHAMBER:

Area of sump chamber: $(15.5\ ft)(22.67) = 351.39\ ft^2$
water at el. 357. $(357 - 348.8) \Rightarrow 8.2$ ft of water

Volume of Water in sump chamber $\Rightarrow 2881.40\ ft^3$
$W_{W_1} = (.0625)(2881.40) \Rightarrow \underline{180.09\ k}$ at 14.42 ft from toe.

3' of water in By-Pass channel: $Vol_I \Rightarrow 4(28)(3) \Rightarrow 336\ ft^3$
$W_{W_2} = (336\ ft^3)(.0625\ k/ft^3) = \underline{21\ k}$ at 3.33 ft from

$Vol_{II} = 4(18.19)(3) \Rightarrow 218.28$
$W_{W_3} = (218.3)(.0625) = \underline{13.64\ k}$ at 14.41 ft from toe

SUBJECT: FORT FAIRFIELD - MAINE LPP
COMPUTATION: PUMPING STATION STABILITY
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-12-87

Stability Analysis [with equipment as shown in layout on previous pages, no other extras (doors, louvers hatch etc.)]

Item	VALUE (K)	Moment arm ft	Moment (K.ft)
W_s	748.38	12.33	9227.53
W_{w_1}	180.09	14.42	2596.90
W_{w_2}	21	3.33	69.93
W_{w_3}	13.64	14.41	196.55
U_1	- 125.02	21.67	- 2709.18
U_2	- 212.79	12.25	- 2606.68
U_{g_1}	- 184.42	3.33	- 614.12
U_{g_2}	- 101.29	15.08	- 1527.45
* P_s	- 132.91 →	7.35	- 976.89
Pump 1	3.0K	17.0	51.0
Pump 2	3.0K	17.0	51.0
Eng 1	3.3 K	11.75	38.78
Eng 2	3.3 K	11.75	38.78
Gate 1	1.3	4.5	5.85
Gate 2	1.3 k	4.5	5.85
oil tank	4.0k	2.5	10.0
Σ	$\Sigma V = 358.79$ $\Sigma H = 132.91$		$\Sigma M = 3857.62$

* Horizontal force.

Stability at Toe el. 345.3

Overturning: $\frac{\Sigma M}{\Sigma V} = \frac{3857.62}{358.79}$

$\Rightarrow 10.75$ ft.

$7.8 < \text{Mid } \frac{1}{3} < 15.7$ } OK within Mid ⅓ 100% in bearing

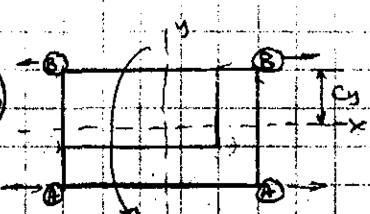
Sliding:

$SF = \frac{\Sigma V \tan\phi + \cancel{cL}^{\to 0}}{\Sigma H} = \frac{358.79 (\tan 28)}{132.91}$

$SF. = 1.44 < 1.5$ OK.

This is a bit low however given that the effects of the soil press. resisting sliding in the East/West direction were not considered, the above factor is found acceptable.

Bearing Pressure: $f_\pm = \frac{\Sigma V}{area} \pm \frac{M_x c_y}{I_x}$ (N/S direction)



$I_x = 33{,}158 \text{ ft}^4$ $c_y = 11.75$ ft Area = 720.51

$f_\pm \Rightarrow \frac{358.79}{720.51} \pm \frac{358.79 (1)(11.75)}{33{,}158} \Rightarrow .498 \pm .127$

along A-A $f_+ = .63$ K/ft²
along B-B $f_- = .37$ K/ft²
} < 4 K/ft² OK.

The following tables provide a detailed breakdown of the quantities and costs of the selected plan. Costs are based on January 1987 prices.

AROOSTOOK RIVER, FORT FAIRFIELD, MAINE

SELECTED FLOOD PROTECTION DIKE PLAN

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
DIKE SECTION	1	JOB	$1,900,000	$1,900,000
PRESSURE CONDUIT	1	JOB	$180,000	$180,000
PUMPING STATION	1	JOB	$460,000	$460,000
STOP-LOGS & RETAINING WALLS	1	JOB	$315,000	$315,000
STORM DRIANAGE SYSTEM	1	JOB	$165,000	$165,000
SEWER RELOCATION	1	JOB	$275,000	$275,000
TOTAL CONSTRUCTION COST				$3,295,000

DIKE SECTION
============

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
SITE PREPARATION	1	JOB	$25,000	$25,000
EXCAVATION	22320	CY	$6	$134,000
STONE PROTECTION	11870	CY	$35	$415,000
GRAVEL BEDDING	4020	CY	$15	$60,000
ROAD GRAVEL	1550	CY	$12	$19,000
DUMPED GRAVEL FILL	16890	CY	$10	$169,000
COMPACTED IMPERV. FILL	65510	CY	$7	$459,000
COMPACTED GARVEL FILL	13680	CY	$12	$164,000
UNDERDRIAN - 6"BCCM	1400	LF	$6	$8,000
UNDERDRIAN - 12"BCCM	1200	LF	$12	$14,000
FILTER MATERIAL - STONE	1740	CY	$30	$52,000
OBSERVATION RISERS	12	EA	$300	$4,000
TOPSOIL SEEDED	9500	SY	$3	$29,000
COMPACTED RANDOM FILL	4000	CY	$4	$16,000
SUBTOTAL				$1,568,000
TOTAL COST INCLUDING 20 % CONTINGENCY				$1,900,000

PRESSURE CONDUIT
================

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
EXCAVATION	2000	CY	$6	$12,000
DRAGBOX	1	ITEM	$4,500	$5,000
CONCRETE STRUCTURAL	45	CY	$300	$14,000
SLIDE GATE W/FLAP GATE	1	ITEM	$20,000	$20,000
STONE BEDDING	110	CY	$30	$3,000
SAND BEDDING	400	CY	$15	$6,000
BACKFILL (EXCAVATED MATL)	1400	CY	$4	$6,000
PAVEMENT SYSTEM	700	SY	$13	$9,000
48" RCP	350	LF	$80	$28,000
MANHOLE (6' ID 16' DEEP)	1	EA	$4,000	$4,000
INLET & OUTLET STRUCTURES	2	JOB	$10,000	$20,000
MANHOLE INLETS	2	EA	$150	$300
STEPS	12	EA	$15	$180
48"DUTILE IORN PIPE	125	LF	$200	$25,000
SUBTOTAL				$152,480
TOTAL COST INCLUDING 20 % CONTINGENCY				$180,000

PUMPING STATION

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
PUMPS ENGINES & GEAR UNITS	1	ITEM	$123,300	$123,000
DISCHARGE PIPE	1	ITEM	$52,300	$52,000
STATION SLUICE GATES	1	ITEM	$39,500	$40,000
HEATING	1	ITEM	$2,000	$2,000
FUELTANK & PIPING	1	ITEM	$3,100	$3,000
LEVEL GAGES	1	ITEM	$5,400	$5,000
VENTILATION	1	ITEM	$2,000	$2,000
EMERGENCY GEN.	1	ITEM	$10,000	$10,000
CONC. WELL STRUCTURAL	45	CY	$300	$14,000
SLUICE GATE (48" GRAVITY P)	1	ITEM	$20,000	$20,000
PUMPING STATION - R. CONC.	180	CY	$300	$54,000
- 8" CUM WALL	1010	SF	$5	$5,000
- 8" BRICK WALL FACE	1260	SF	$6	$8,000
ROADWAY GURD POSTS	1	JOB	$3,000	$3,000
CHAIN LINK FENCING	1	JOB	$3,000	$3,000
6'X10' DOOR	1	EA	$1,500	$2,000
48" DUCTILE IORN PIPE	140	LF	$200	$28,000
STONE BEDDING	150	CY	$30	$5,000
SAND BEDDING	150	CY	$15	$2,000
STEEL SHEET PILING	180	SF	$20	$4,000
EXCAVATION	350	CY	$4	$1,000
SUBTOTAL				$386,000
TOTAL COST INCLUDING 20 % CONTINGENCY				$460,000

STOP-LOGS & RETAINING WALLS

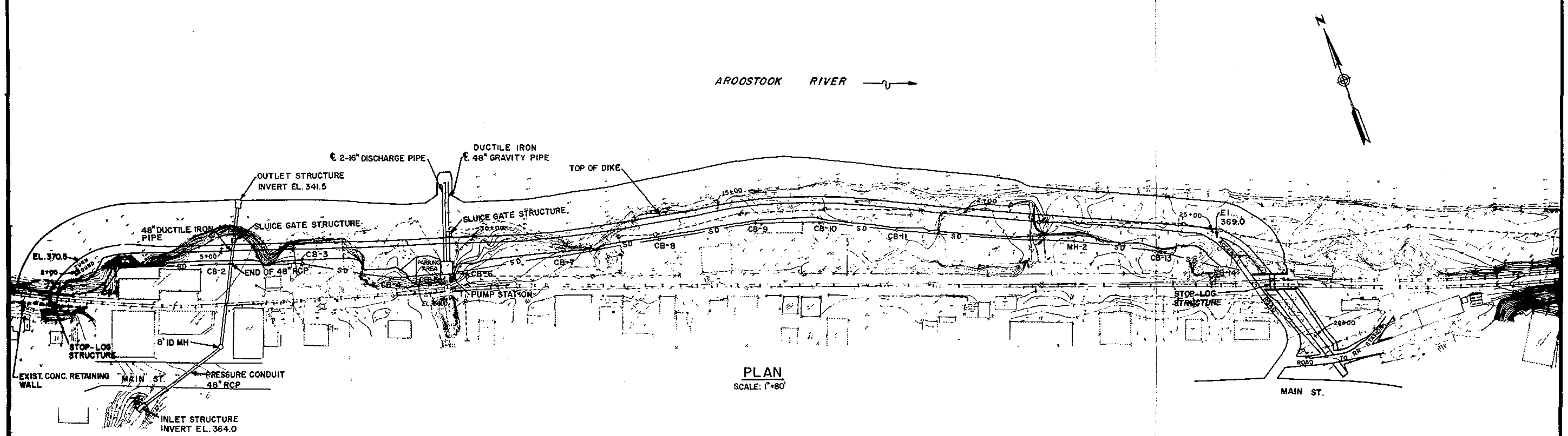
ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
EXCAVATION	1260	CY	$6	$8,000
T-WALL	135	CY	$300	$41,000
STOP-LOG (UPSTREAM)	108	CY	$300	$32,000
8"X10" WHITE OAK LOGS 18.5'	12	EA	$250	$3,000
SAND BAGS	135	EA	$10	$1,000
100'SINGLE RR TRACK REMOVE & RESTOR	1	JOB	$10,000	$10,000
COMPACTED GRAVEL	340	CY	$12	$4,000
GRAVITY WALL	320	CY	$300	$96,000
STOP-LOG (DOWENSTREAM)	65	CY	$300	$20,000
8"X8" WHITE OAK LOGS 14'	22	EA	$150	$3,000
W10X22 STL CENTER POST 11'	1	EA	$500	$1,000
120' DOUBLE RR TRACK REMOVE & RESTOR	1	JOB	$25,000	$25,000
REPLACE RR SWITCH	1	JOB	$20,000	$20,000
SUBTOTAL				$264,000
TOTAL COST INCLUDING 20 % CONTINGENCY				$315,000

STORM DRAINAGE SYSTEM

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
EXCAVATION	3600	CY	$6	$22,000
STONE BEDDING	220	CY	$30	$7,000
SAND BEDDING	650	CY	$15	$10,000
BACKFILL (EXCAVATED MATL)	2700	CY	$4	$11,000
30" ID RCP	420	LF	$40	$17,000
24" ID RCP	330	LF	$24	$8,000
18" ID RCP	830	LF	$18	$15,000
12" ID RCP	780	LF	$10	$8,000
MANHOLES 4'ID 8' DEEP	15	EA	$1,500	$23,000
CATCH BASINS & FRAMS	15	EA	$500	$8,000
MANHOLE INLETS	30	EA	$150	$5,000
TIE IN EXIST 12" DRAIN	2	JOB	$250	$1,000
TOPSOIL SEEDED	2200	SY	$3	$7,000
SUBTOTAL				$142,000
TOTAL COST INCLUDING 20 % CONTINGENCY				$165,000

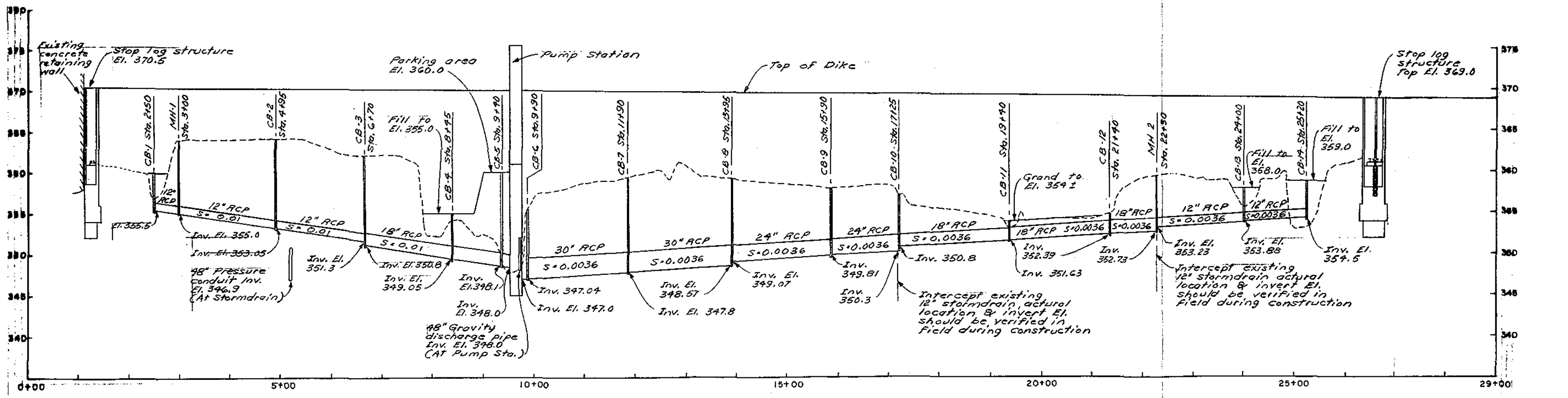
SEWER RELOCATION

ITEM	QUANTITY	UNITS	UNIT PRICE	TOTAL COST
EXCAVATION	4000	CY	$6	$24,000
SAND BEDDING	400	CY	$15	$6,000
STONE BEDDING	150	CY	$30	$5,000
BACKFILL (EXCAVATED MATL)	3600	CY	$4	$14,000
DRAGBOX (TRENCH CONST)	1	ITEM	$4,500	$5,000
5' ID MANHOLE (13'DEEP)	8	EA	$3,500	$28,000
18" RCP	1400	LF	$18	$25,000
TIE IN EXIST SEWERS	17	EA	$250	$4,000
ABONDON EXIST 16" PLUG	21	EA	$50	$1,000
BACKFILL MANHOLES	9	EA	$100	$1,000
TOPSOIL SEEDED	2200	SY	$3	$7,000
PERMANENT SHEETING NEAR RR	4500	SF	$25	$113,000
SUBTOTAL				$233,000
TOTAL COST INCLUDING 20 % CONTINGENCY				$275,000



PLAN

SCALE: 1"=80'



PROFILE OF STORMDRAIN AND TOP OF DIKE

SCALE: VERT. 1" = 5'
HORIZ. 1" = 80'

1" = 80'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN

LOCAL PROTECTION PROJECT

AROOSTOOK RIVER

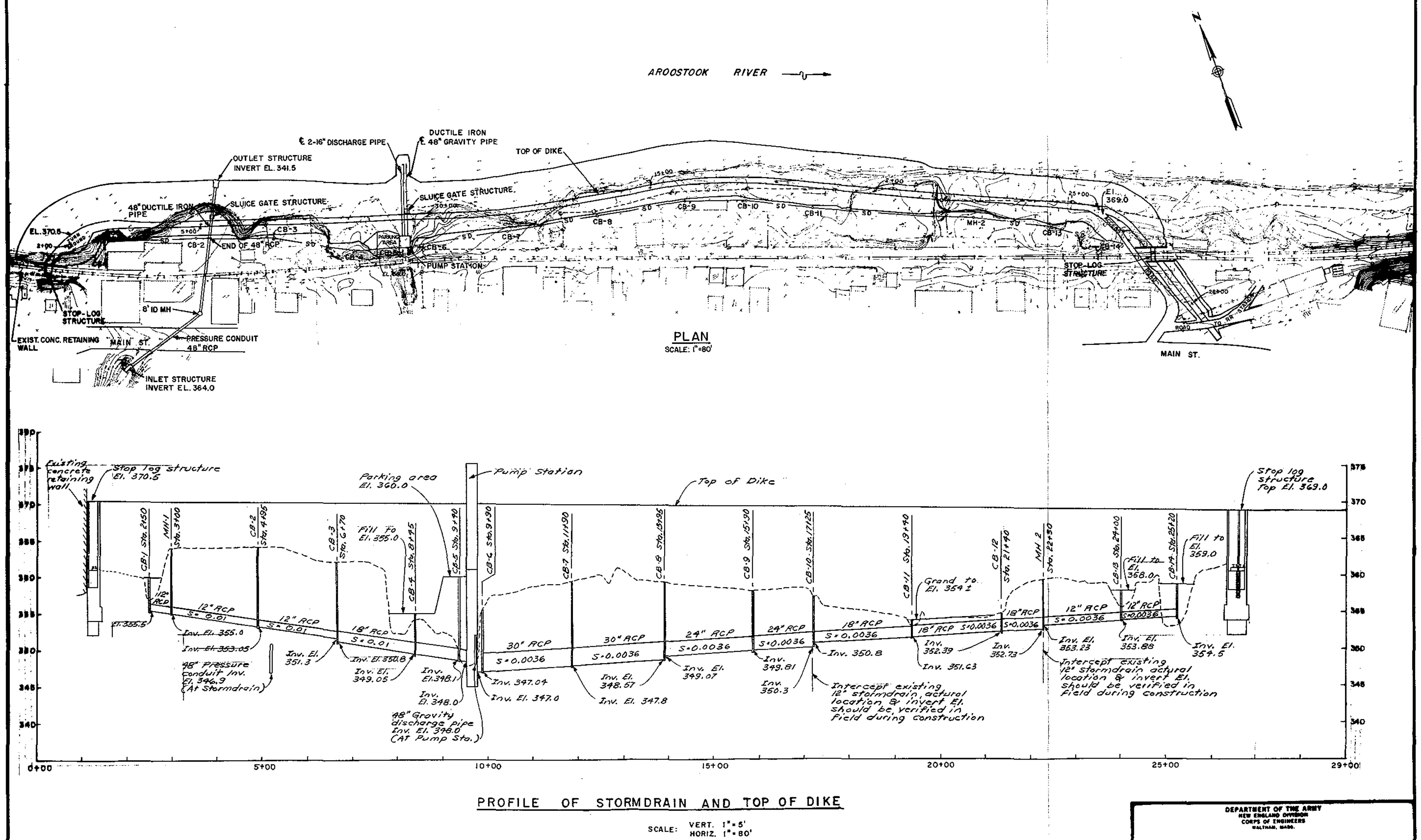
FORT FAIRFIELD, MAINE

PLAN AND PROFILE OF STORMDRAIN

SCALE: AS SHOWN

DATE: MAR. 1987

PLATE C-1



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CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN

LOCAL PROTECTION PROJECT

AROOSTOOK RIVER

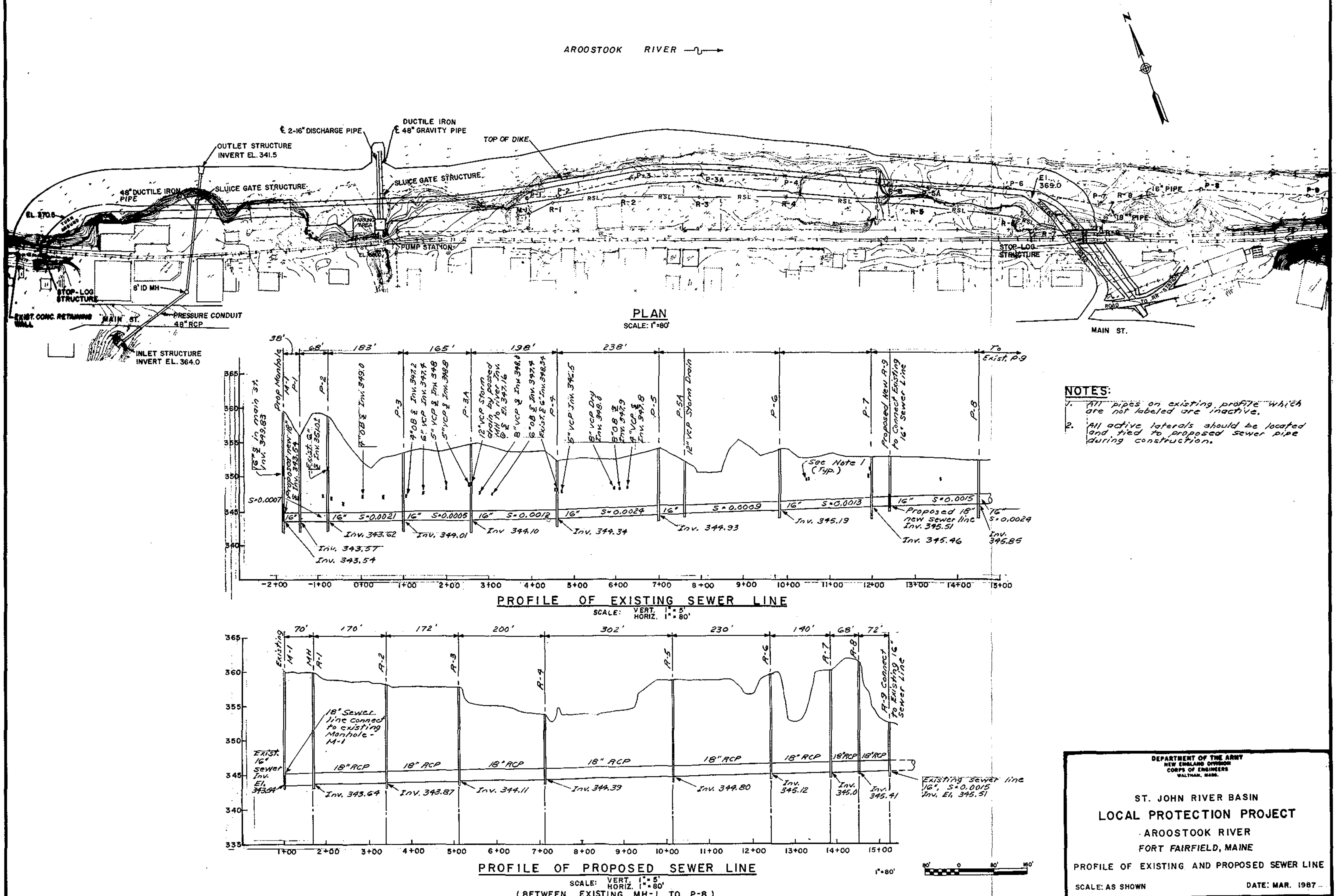
FORT FAIRFIELD, MAINE

PLAN AND PROFILE OF STORMDRAIN

SCALE: AS SHOWN

DATE: MAR. 1987

PLATE C-1



NOTES:

1. All pipes on existing profile which are not labeled are inactive.
2. All active laterals should be located and tied to proposed sewer pipe during construction.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
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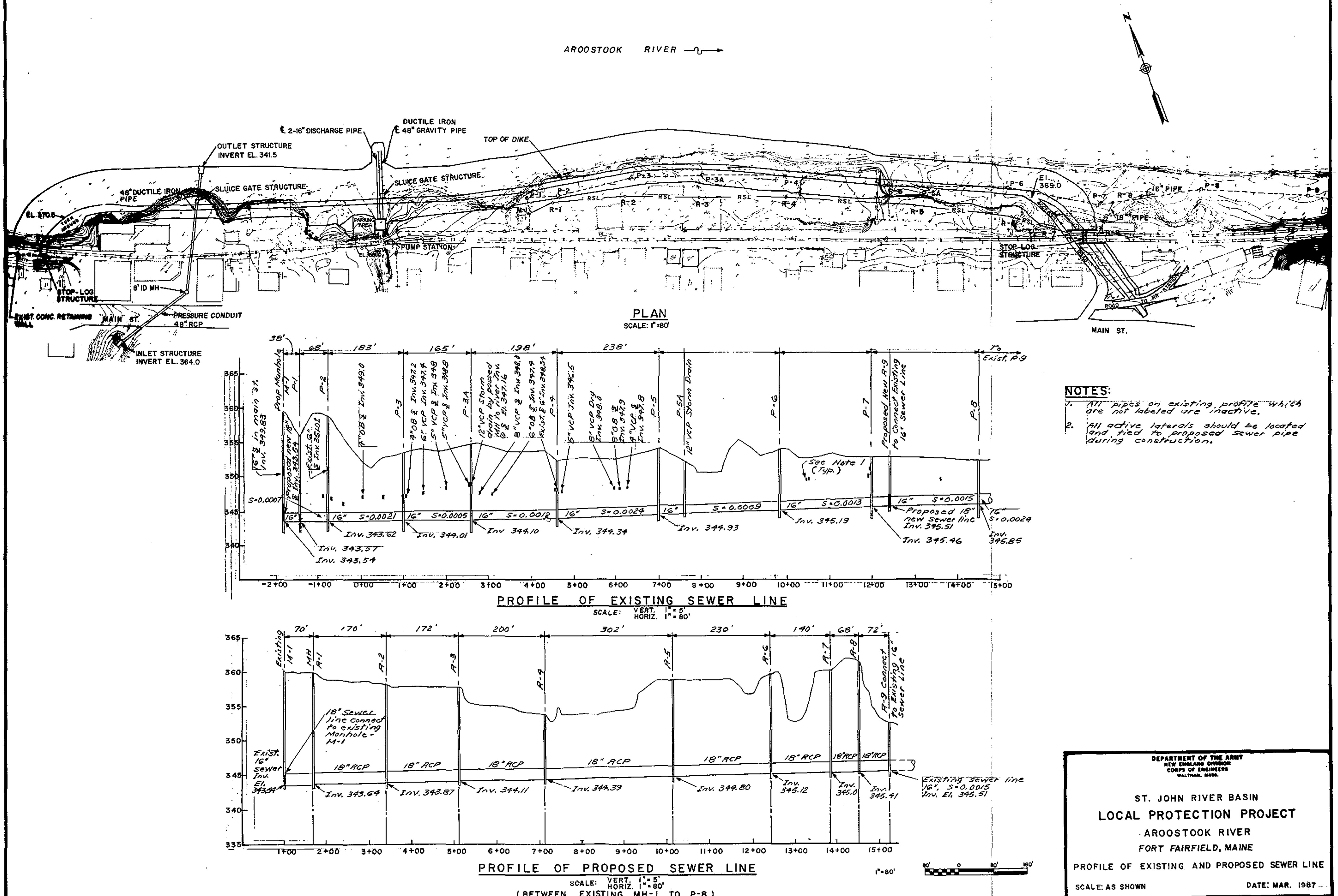
WALTHAM, MASS.

ST. JOHN RIVER BASIN
LOCAL PROTECTION PROJECT
AROOSTOOK RIVER
FORT FAIRFIELD, MAINE
PROFILE OF EXISTING AND PROPOSED SEWER LINE

SCALE: AS SHOWN
DATE: MAR. 1987

PLATE C-2



NOTES:

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2. All active laterals should be located and tied to proposed sewer pipe during construction.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN

LOCAL PROTECTION PROJECT

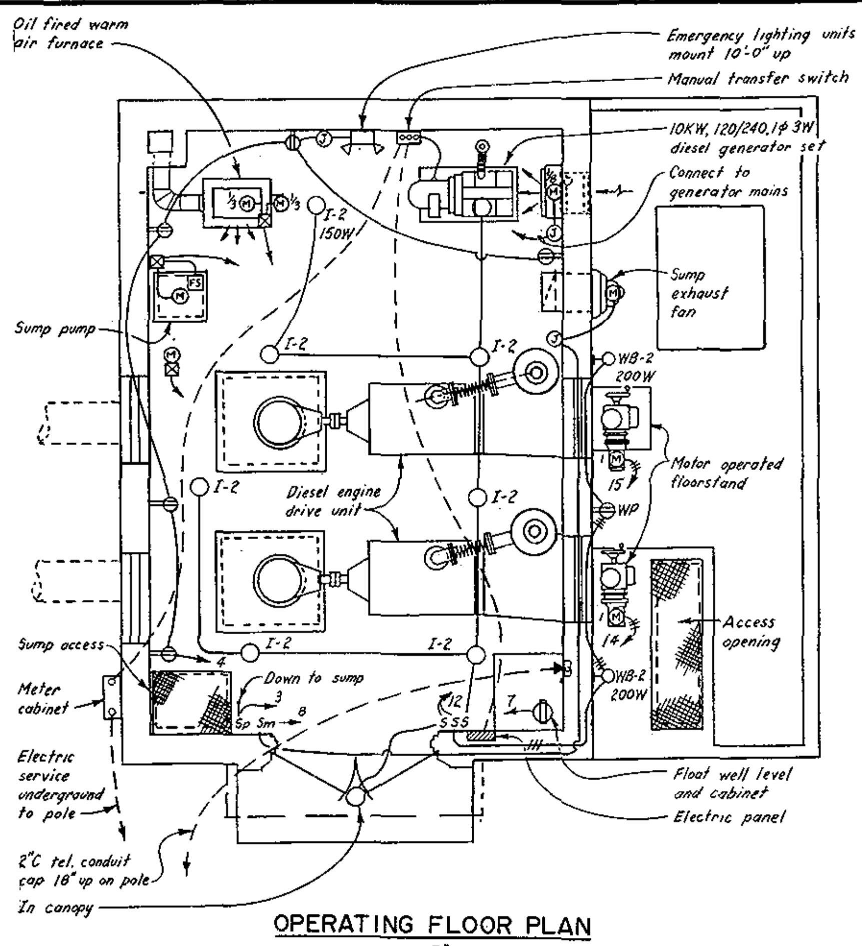
AROOSTOOK RIVER

FORT FAIRFIELD, MAINE

PROFILE OF EXISTING AND PROPOSED SEWER LINE

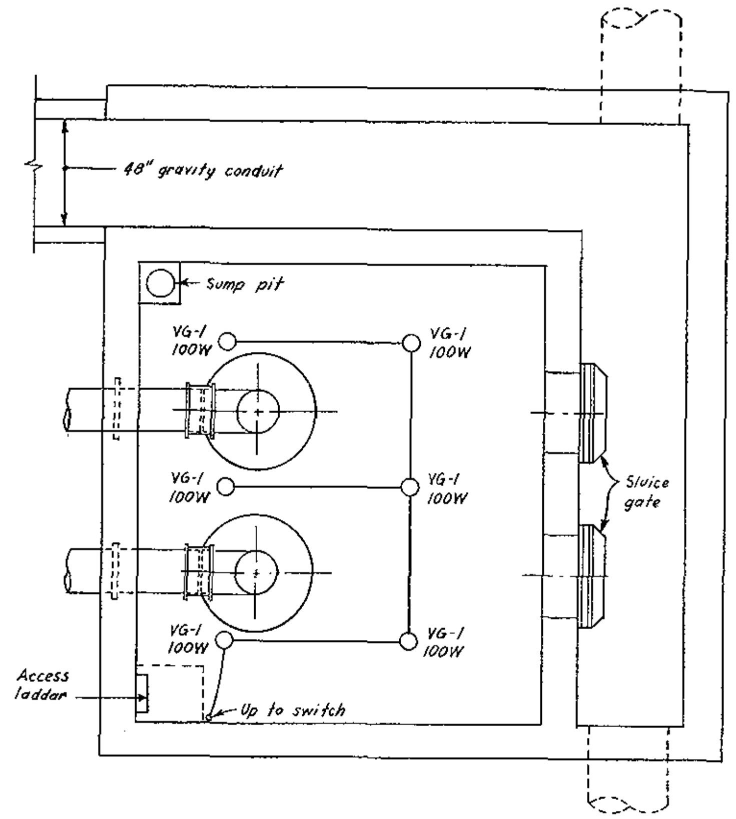
SCALE: AS SHOWN DATE: MAR. 1987

PLATE C-2



OPERATING FLOOR PLAN

SCALE: 3/8" = 1'-0"



SUMP PLAN

SCALE: 3/8" = 1'-0"

PANEL

120/240V, 1 1, 3 WIRE, 100AMP. MAIN BREAKER

CIRCUIT NUMBER	BREAKER SIZE AMPS	NUMBER OF POLES	SERVES	CONNECTED LOAD KVA
1	20	1	Lighting	1.2
2	20	1	Spare	–
3	20	1	Furnace	2@ 1/3
4	20	1	Receptacles	1.0
5	20	1	Sump lighting	0.8
6	20	1	Float well receptacle	0.2
7	20	2	Spare	1
8	20	1	Sump exhaust fan	1/3
9	20	1	Spare	–
10	20	1	Spare	–
11	20	1	Grease unit	1/3
12	20	1	Exterior lighting	0.6
13	20	2	Sump pump	1
14	20	2	Floor stand No.1	1
15	20	2	Floor stand No.2	1
16	20	2	Sluice gate (Gravity Outlet)	2

LEGEND

SYMBOL	DESCRIPTION
	Ceiling outlet
	Wall outlet
	Junction box
	Duplex convenience outlet - WP indicates weatherproof
	Motor - HP as indicated
	Emergency lighting unit
	Panel
	Controller
	Telephone outlet, 1'-0" up
	Branch circuit concealed in ceiling or wall
	Branch circuit in floor
	Home run to panelboard. Note: Any circuit without further designation indicates a two-wire circuit. For a greater number of wires indicate as follows (3 wires) (4 wires) etc.
S	Single pole switch, 4'-0" up
FS	Float switch

SAFETY PAYS

GRAPHIC SCALES

3/8" = 1'-0" 4 0 4

1" = 10' 10 0 10 20

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY F. T. DR. BY S.K. CK. BY

SUBMITTED: CHIEF, GEN. ENG. SECTION

APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH

REVIEWED: PROJECT MANAGER

APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FORT FAIRFIELD, MAINE

LOCAL PROTECTION PROJECT
PUMPING STATION
ELECTRICAL-PLANS AND SCHEDULE

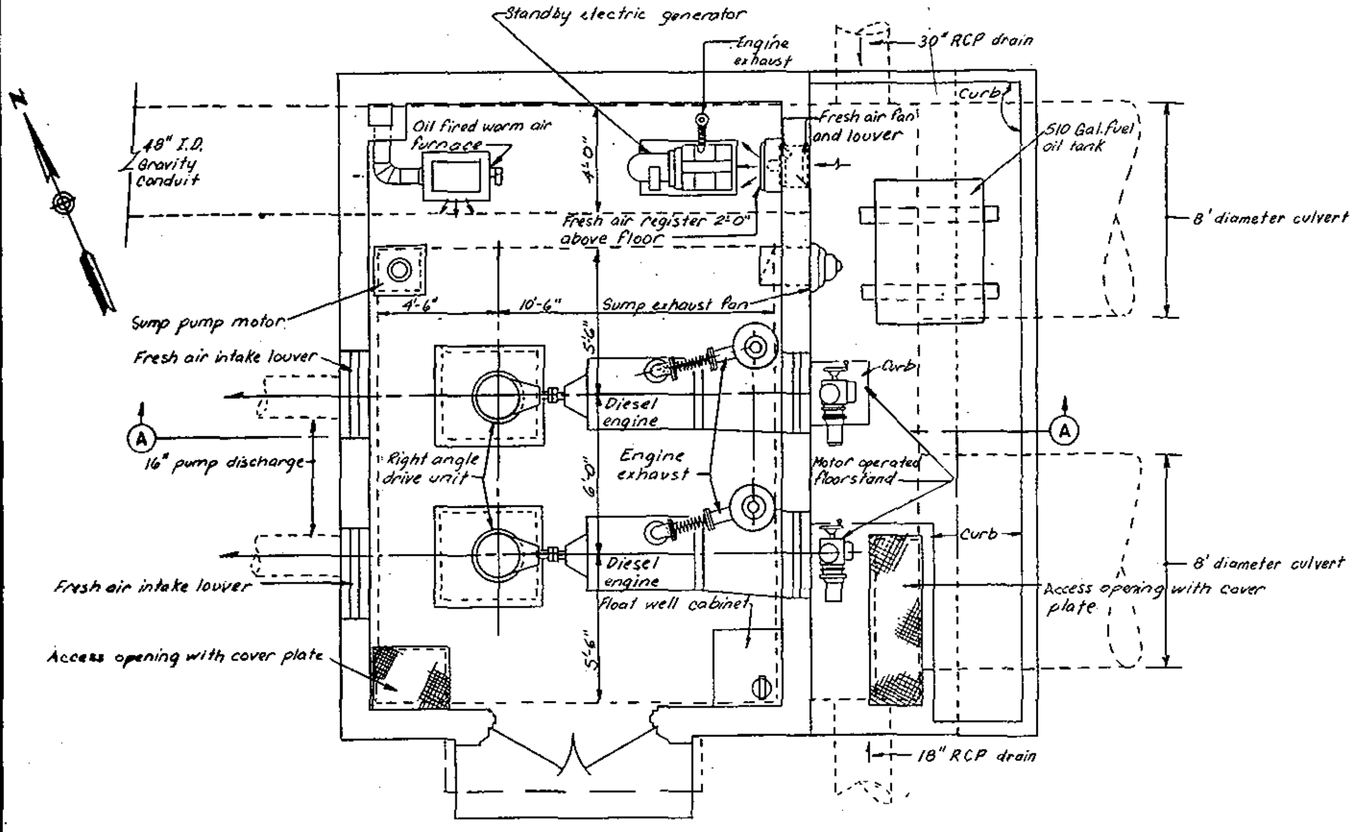
AROOSTOOK RIVER MAINE

APPROVED DATE
CHIEF, ENGINEERING DIVISION

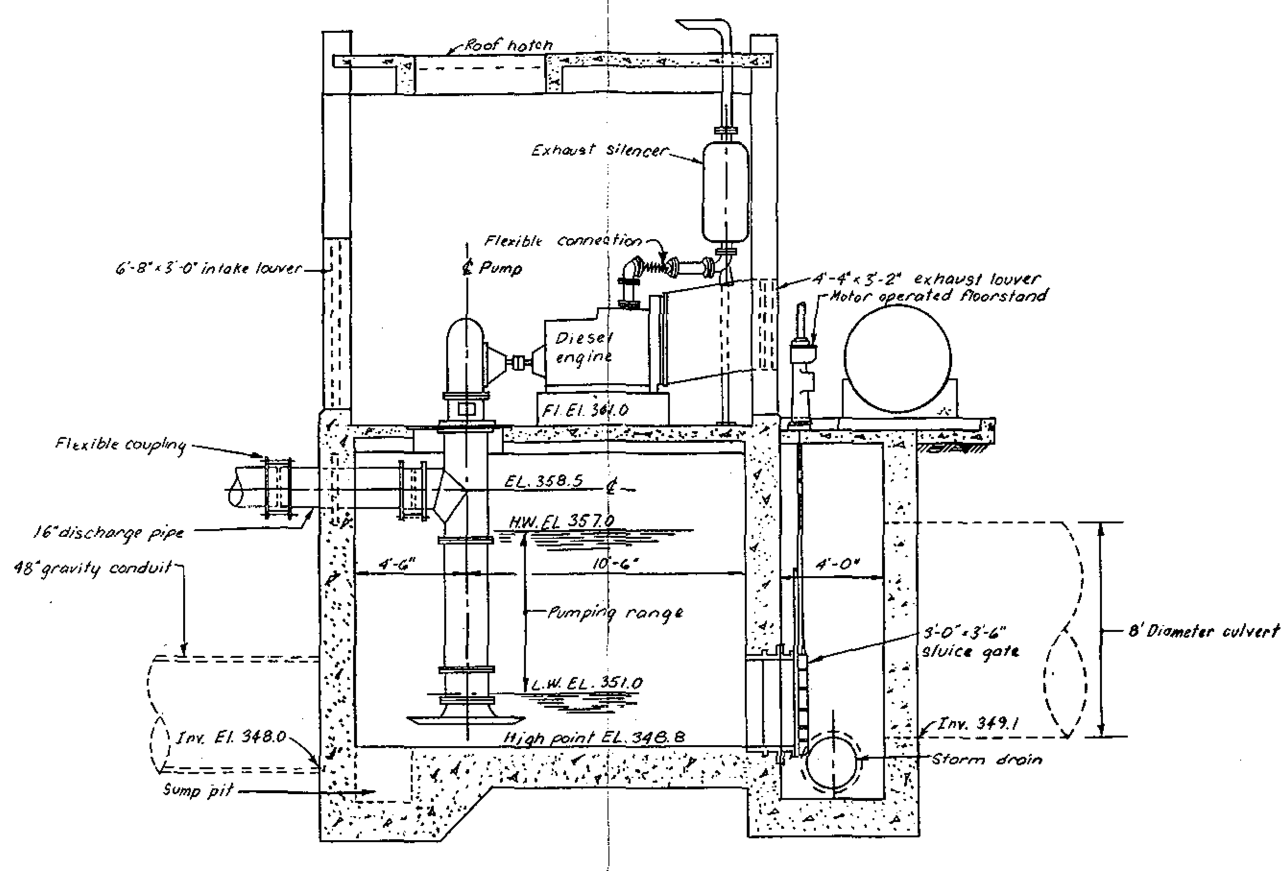
SCALE: AS SHOWN SPEC. NO.

DRAWING NUMBER

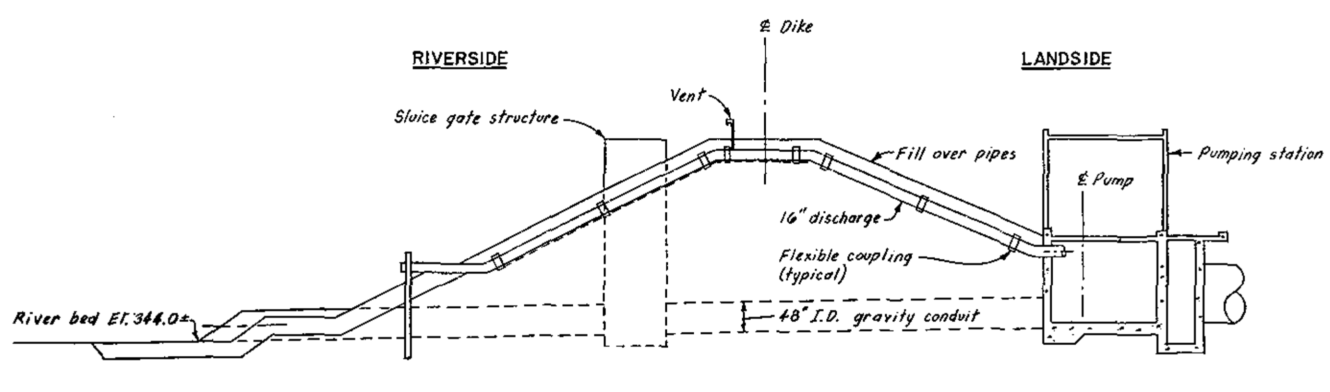
SHEET



FLOOR PLAN
SCALE: 3/8" = 1'-0"



SECTION A-A
SCALE: 3/8" = 1'-0"



SECTION AT DIKE
SCALE: 1" = 10'

GRAPHIC SCALES
3/8" = 1'-0" 4' 0 4'
1" = 10' 10' 0 10' 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ST. JOHN RIVER BASIN
LOCAL PROTECTION PROJECT
AROOSTOOK RIVER
FORT FAIRFIELD, MAINE
PUMPING STATION
MECHANICAL - PLANS AND SECTION

SCALE: AS SHOWN DATE: MAR. 1967

PLATE C-4

SECTION D

SOCIAL AND ECONOMIC ANALYSIS

FORT FAIRFIELD, ME
ECONOMIC ANALYSIS

Table of Contents

Introduction

The purpose of this section is to measure the beneficial contributions to national economic development that are associated with the water resources improvement plans for the Fort Fairfield floodplain. The extent to which the flood control needs of the area are met by the plans will be determined by estimating the dollar value of inundation reduction benefits produced by the plans. Explanatory rationale and supporting documentation will be presented. The measure of each plan's economic justification is the benefit-cost ratio, which is calculated by dividing the dollar value of the total annual benefits to be realized over the plan's economic life by the annual charges for the plan's total cost. A benefit-cost ratio of 1.0 or greater is necessary for Federal participation in water resources improvement projects. Simply, one dollar's worth or more of flood reduction benefits is required for each dollar to be expended on project construction. If more than one plan of improvement has a benefit-cost ratio greater than 1.0 then the plan with the greatest amount of net benefits (ie. total annual benefits minus total annual costs) is chosen. The plan which maximizes net benefits allocates limited resources in the most efficient manner and provides the greatest return on public investment. The analysis contained in this section was performed in accordance with Economic Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies, Water Resources Council, 1983. Dollar values stated in this section reflect the December 1986 price level. Discounting and amortization was performed at 8-7/8 percent, the current interest rate for Federal water resources improvement project evaluation.

Socio-Economic Setting

The town of Fort Fairfield is located in Aroostook County, Maine. This county contains more than 20 percent of Maine's land area but only 8 percent of its people. The rural nature of the county is indicated by its population density per square mile of 13.6 compared to 35.3 statewide. The population of Fort Fairfield is 4,376 (1980 U.S. Census). Both the town and Aroostook County have experienced population declines over the past 20 years, while the state of Maine population overall has been growing since 1940.

TABLE 1
POPULATION TRENDS 1960 - 1980

	1960	1970	1980	% Change 1960-1970	% Change 1970-1980
Fort Fairfield	5,876	4,859	4,376	-17.3	-9.9
Aroostook County	106,064	94,078	91,331	-11.3	-2.9
State of Maine	969,300	993,700	1,124,660	+2.5	+13.2

The population declines in Fort Fairfield and Aroostook County can be traced to fewer agricultural jobs due to mechanization and a decline in competitive market position in the potato industry. Other employment sectors are also not providing job opportunities in sufficient numbers to halt emigration of job-seekers from the county. The increases in population for the state of Maine reflect the growth and development in the southern counties, especially the seacoast communities and those nearby.

The economic well-being of Fort Fairfield inhabitants can be measured by examining per capita income, median family income and percentage of families at or below the poverty level.

TABLE 2
INDICATORS OF ECONOMIC WELL-BEING

	Per Capita Income	Median Family Income	% of Families below Poverty Level
Fort Fairfield	$4,460	$14,022	10.2%
Aroostook County	$4,826	$13,924	13.3%
State of Maine	$5,768	$16,167	9.8%

Fort Fairfield and Aroostook County obviously have income measures below statewide figures because of their rural nature and lack of a strong industrial base. However, Fort Fairfield families do fare slightly better than average county families in terms of median income. Also, while the town's poverty level percentage is nearly that of the state it is 3 percentage points lower than that of Aroostook County.

According to the 1980 U.S. Census, the Fort Fairfield labor force was 1,735 people of which 1,598 were employed. Of the employing industries, services accounted for the largest share (23%) mostly in health and education. Other major employing industries are: manufacturing (17.5%), agriculture (13.5%), retail trade (13.7%), public administration (7.7%) and construction (6.3%). The majority of employed persons are private wage and salary workers (63%) with the remainder working for Federal, state or local government (26%) or self-employed (11%).

Study Area

The actual study area is comprised of approximately 25 acres along both sides of Main Street in the commercial district of Fort Fairfield. Main Street is located adjacent to the Aroostook River and its low-lying one-half mile stretch between Peterson's Garage and the Canadian Pacific Railroad Office has been the scene of many floods. Most of the flooding occurs during the springtime because of snowmelt and in many instances is exacerbated by ice jams.

The character of the study area is mostly commercial, however, there is a concentration of senior citizen housing units. There are 30 commercial structures in the area which house 41 separate commercial activities, 4 fraternal organizations and one government agency. Of these 30 structures, 5 have apartments on the second story and one has a total of 25 apartments on its second and third stories. There is only one traditional two-story, two-family house in the study area, but there are two senior citizen housing complexes. The first, Northern House, is a 3-story structure which contains 26 apartments. The second is the Fields Lane Senior Citizen Complex and is operated by the Housing Authority of Fort Fairfield. The complex is a campus type layout with 9 detached structures accounting for a total of 40 units plus a community center. Rounding out the structural inventory of the study area are two government buildings, one the U.S. Post Office and the other the Fort Fairfield Municipal Building which is occupied by town offices, the Police Department and Fire Department.

Valuation of Properties in the Study Area

In November 1986 the Town of Fort Fairfield provided the total value, based on Town Assessor's records, of the properties in the Main Street study area. The value of land is $502,860 and buildings is $3,641,000 for a total value of $4,143,860. Town officials indicate that this figure is roughly 94 percent of current market value.

Flood Damage Surveys

Flood damage surveys are performed at the start of every Corps of Engineers flood control study in order to determine the need for improvements by estimating the magnitude of potential flood-related losses. These losses are estimated, at each flood-prone structure and site, starting at the elevation at which flooding and damage begins up to the elevation of floodwater associated with a very rare event such as the 500 year storm. Damages are estimated in one-foot increments between these two limits. The categories of these losses are: commercial, industrial, residential, agricultural and public. The two types of losses are physical and non-physical. Physical losses relate to grounds, site, structure, contents, utilities and clean-up. Non-physical losses are those additional induced costs which result from loss of use of a flooded structure. Residential non-physical losses are the costs of food, lodging and necessities while unable to use one's residence. For commercial and industrial firms non-physical losses are measures such as lost income and profit while shut down plus the cost of temporary quarters and services. In addition to the structure-related loss categories above, the flood damage survey estimation process also covers two general loss categories: (i) cost of emergency services and (ii) damages and costs to transportation, communication and utility systems.

The first flood damage survey of the Fort Fairfield study area was performed in October 1977 by a private consulting engineering firm as part of the larger St. John River Basin study. In October 1982, damage evaluators from the New England Division performed a major on-site update. Updates have been performed recently in November 1985 and December 1986 to document improvements which have taken place in the study area.

Recent and Planned Improvements in Study Area

In 1985 the State of Maine awarded a Community Development Block Grant in the amount of $820,000 to fund the 2-year Fort Fairfield Downtown Revitilization Project. Under this project certain commercial buildings were renovated and expanded and some older buildings were razed. Private investment in the study area was also made during 1986. The Irving Oil Co. constructed a large gas station, grocery store and liquor store. In 1987, the State of Maine, Department of Transportation plans to completely excavate and construct a new roadway and sidewalks for Main Street in the study area. Other improvements for Main Street scheduled for 1987 are: (i) the installation of 125 new street lights, (ii) installation of a new 8 inch sanitary sewer line (1600 linear feet) with manholes and service extensions and (iii) reinforcement of the existing telephone system, both underground and aerial, along Main Street by New England Telephone. The total cost for these 4 scheduled improvements is $1,500,000.

Susceptibility to Flooding

One indicator of an area's susceptibility to damage from flooding is the relationship of the first floor elevation of structures in the floodplain to the elevation of floodwaters from certain events. First floor elevations were obtained for all floodplain structures by a field survey crew and potential flood elevations were obtained from an "elevation vs. frequency" curve produced by the Water Control Branch (Hydrologic Engineering Section) of the New England Division. The summary table below shows the relationship between flood elevation, frequency and number of structures affected. The salient point of the table is that even a storm of 10 year frequency will produce a flood level that will cover the first floor of 25 of the 43 floodplain structures.

TABLE 1
STRUCTURES SUSCEPTIBLE TO FIRST-FLOOR FLOODING
FORT FAIRFIELD STUDY AREA

Event (year)	Annual % Chance of Occurrence	Flood Elevation (NGVD)	Structures w/ First Floor Flooding Number	% of Total
100 yr.	1%	367.3'	37	86%
50 yr.	2%	366.4'	33	77%
10 yr.	10%	363.9'	25	58%

Recurring Losses

Recurring losses are those potential flood related losses which are expected to occur at various stages of flooding under present day development conditions. Table 2 below displays the dollar value of potential flood-related losses, by damage category, that are estimated to occur if that specific flooding event were to occur today.

TABLE 2
RECURRING FLOOD LOSSES
FORT FAIRFIELD STUDY AREA

Category	10 Year Event (el. 363.9')	50 Year Event (el. 366.4')	100 Year Event (el. 367.3')	500 Year Event (el. 369.2')
Properties	$1,107,000	$3,592,000	$4,678,000	$6,795,000
Emergency Costs	14,800	24,600	33,800	53,200
Downtown Roads	20,000	239,400	273,100	273,100
Railroads	87,300	174,500	174,500	174,500
Total Losses	$1,229,100	$4,030,500	$5,159,400	$7,295,800

Annual Losses

Recurring losses, discussed above, are informative inasmuch as they relate the dollar value of flood losses to specific depths of flooding, however they don't offer any information as to what the chances are of those flooding depths occurring in any given year. For the purpose of determining the severity of potential flooding the statistical concept of expected value is employed. For flood control studies the term used to measure the severity of potential flooding on an annual basis is "annual losses." Annual losses are calculated by integrating two sets of data: (i) recurring losses displayed in one-foot increments of flood depth from start of damage to the 500 year storm elevation and (ii) the estimated annual percent chance that flooding will reach each specific elevation for which recurring losses were estimated. Recurring losses are obtained by the flood damage survey process and the annual percent chance of occurrence for each event is obtained form a stage-frequency curve. This curve, estimated by the Hydrologic Engineering Section at NED, displays flood stages on the X-axis and the annual percent chance of reaching that stage on the Y-axis. Annual losses are computed for each event from the one that first causes damage to the 500 year event. Losses for all events are aggregated and this total estimate of expected annual losses represents the degree of flooding severity in the study area. The effectiveness of each alternative plan that is formulated for flood reduction is measured by the extent to which it reduces annual losses. Annual losses, by category, for the Fort Fairfield study area are displayed in Table 3.

TABLE 3
ANNUAL LOSSES
FORT FAIRFIELD STUDY AREA

Category	Annual Losses
Properties	$398,400
Emergency Costs	2,800
Downtown Roads	12,300
Railroads	47,000
Total	$460,500

Economic Benefit Analysis

Benefits from plans for reducing flood hazards accrue primarily through the reduction in actual or potential damages associated with land use. Benefits fall into three categories reflecting different responses to a flood hazard reduction plan. The inundation reduction benefit accrues when land use is the same with or without the plan and is defined as the increased net income generated by that use. The intensification benefit also accrues when land use is unchanged and is defined as the increase in net income based on a modification of the method of operation by floodplain occupants because of the plan. The location benefit accrues when an activity is added to the floodplain because of a plan and is measured as the difference between aggregate net incomes in the economically affected area with and without the plan.

Under the "with plan" condition for the Fort Fairfield study area, land use is projected to remain essentially the same. Since the area is the center of commercial activity and has a considerable number of permanent elderly housing units, it is projected that these functions will continue into the foreseeable future. This projection is nearly irrefutable based on the public and private investments in the area's infrastructure and commercial activities during 1985 to 1987. There probably will be modifications to existing activities and development on some of the few vacant lots, with the plan, but it is not expected to be on a large enough scale to significantly affect future losses and benefits. Therefore, benefits which accrue to the improvement plans will be measured under the category of inundation reduction only.

Inundation Reduction Benefit

The increase in net income that accrues under this category is measured by the decrease in the dollar value of outlays associated with reduced flood losses. The national economic development (NED) objective is satisfied if an improvement plan produces the beneficial impact of reducing annual losses.

Improvement Plans Evaluated

Three improvement plans, each offering a different level of protection, were evaluated. All three plans involve a 3000 foot long earthen dike which would extend from just upstream of Peterson's Repair Garage downstream to the Canadian Pacific Railroad Office. The plans to be evaluated offer flood protection against the following 3 events: (i) 500 year, (ii) 100 year and (iii) 50 year.

Benefit Estimation

Benefits for inundation reduction were calculated based on the flood elevation corresponding to each event. The top elevation of each dike plan is that flood elevation plus and additional 3 feet of freeboard to account for wave run-up and wind effects. Corps of Engineers regulations allow benefits to be taken up to the top of the dike plus 50 percent (1.5 feet) of the freeboard range. The benefits to each plan are the summation of annual losses prevented by the dike taken to an elevation 1.5 feet below the absolute top of dike including freeboard. The benefits for each plan are enumerated in Table 4.

TABLE 4
ANNUAL BENEFITS - INUNDATION REDUCTION
FORT FAIRFIELD STUDY AREA

	Annual Inundation Reduction Benefits		
	Level of Protection		
Category	500 Year (el. 369.5')	100 Year (el. 368')	50 Year (el. 367')
Properties	\$387,400	\$362,200	\$327,000
Emergency Costs	2,800	2,600	2,300
Downtown Roads	11,900	10,700	8,900
Railroads	46,600	46,000	44,800
Total	\$448,700	\$421,500	\$383,700

Reduced Pumping Costs

A second type of flood related cost that will be reduced by the dike plan is the increased pumping costs at the Fort Fairfield Sewage Treatment Plant during times of flooding. There is a sewer pipe which runs along the entire length of the site where the dike would be constructed. This pipe would require relocation closer to Main Street, away from the river bank if the dike were to be constructed. In order to determine if economic benefits would accrue to this relocation, the manager of the Fort Fairfield Utilities District was interviewed. The pipe does not currently sustain direct damage from flooding or erosion. It was installed in 1976, is made of PVC, is buried 13 to 17 feet below ground and has an expected life of 60 years. However, during periods of flooding at the pipe's location, especially in springtime, inflow and infiltration of floodwaters into the pipe occurs at manholes and around some pipe joints. Pumping at the treatment plant increases dramatically from an average of 0.4 MGD to 1.5 MGD during times when floodwaters enter the system and continues at the elevated rate for 2 weeks after flooding subsides. There are two negative effects caused by this inflow. First, the pumping system is overburdened and must pump flood water that doesn't need treatment. Because of this, untreated sewage also gets pumped into the river. The Utilities District is currently under a consent decree from the Maine Department of Environmental Protection to control the inflow. Secondly, the increased volume which needs to be pumped during times of flooding increases the pumping costs. Under the with-plan condition, the section of pipe where inflow and infiltration occurs will be relocated to the inside of the dike, closer to Main Street and further away from the riverbank. The manger of the Utilities District indicates that this relocation of the pipe should solve the inflow/infiltration problem as the manholes will be in the flood protection area. The pumping plant will not be overburdened, pumping costs will remain at normal levels, and untreated sewage will not be pumped into the river, thereby keeping the Utility District in compliance with its State and Federal licenses. The benefit to be realized with the project is estimated to be $2,000 annually in reduced pumping and associated repair costs.

Reduction in Flood Insurance Overhead Costs

A cost of floodplain occupancy is flood insurance overhead costs. This administrative cost is national in nature and will be eliminated with the 500 year and 100 year dike improvement plans. The 1986 overhead cost per policy is $67 and an estimated 36 policies are in effect in the study area. With the improvement plan the annual benefit is $2,400.

Summary of Benefits

The annual benefits expected to accrue under each of the 3 flood protection plans are exhibited in Table 5 below.

TABLE 5
SUMMARY OF ECONOMIC BENEFITS
FORT FAIRFIELD FLOOD REDUCTION PLANS

Category	500 Year Protection	100 Year Protection	50 Year Protection
	Annual Benefits		
Inundation Reduction:			
Properties	$387,400	$362,200	$327,700
Emergency Costs	2,800	2,600	2,300
Downtown Roads	11,900	10,700	8,900
Railroads	46,600	46,000	44,800
Reduced Pumping Costs (Sewage Treatment Plant)	2,000	2,000	2,000
Reduction in Flood Insurance Overhead Costs	2,400	2,400	-
TOTAL BENEFITS	$453,100	$425,900	$385,700

Economic Justification

The ultimate purpose of the economic analysis is to compare the benefits estimated for each plan to the annual costs of plan implementation in order to determine the benefit-cost ratio which is the measure of economic justification and indicator of Federal participation.

TABLE 6
ECONOMIC EVALUATION OF PLANS

	500 Year Protection	100 Year Protection	50 Year Protection
Total Annual Benefits	$453,100	$425,900	$385,700
Total Annual Costs	$435,000	$385,000	$361,000
Benefit-Cost Ratio	1.04	1.11	1.07
Net Benefits	$18,100	$41,900	$24,700

SECTION E

REAL ESTATE

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

424 TRAPELO ROAD

WALTHAM, MASSACHUSETTS 02154-9149

PRELIMINARY ESTIMATE OF REAL ESTATE COSTS
FORT FAIRFIELD LOCAL PROTECTION PROJECT
AROOSTOOK RIVER, FORT FAIRFIELD, MAINE

May 1987

PREPARED BY: Robert P. Abbott
ROBERT P. ABBOTT
Staff Appraiser

REVIEWED &
APPROVED BY: William D. Brown, Jr.
WILLIAM D. BROWN, JR.
Chief, Appraisal Branch

TABLE OF CONTENTS

1. PURPOSE

The purpose of this report is to estimate the preliminary real estate cost associated with the Fort Fairfield Flood Control Project located in Fort Fairfield, Aroostook County, Maine.

2. INSPECTION OF THE REAL ESTATE

The properties affected by the proposed project were inspected/viewed in the field in May, 1987, by Staff Appraiser Robert P. Abbott, of New England Division, Corps of Engineers.

3. LOCATION AND AREA DATA

Fort Fairfield is located near the Canadian Border in northeastern Maine on the southern bank of the Aroostook River. The Great Atlantic & Pacific Tea Co. operates a $3.6 million dollar potato and pea processing plant in Fort Fairfield that employs approximately 200 people.

The area is serviced by U.S. Highway IA and State Highway 167 in addition to two railroads, the Canadian Pacific and the Bangor & Aroostook. At present, Fort Fairfield is a progressive agricultural community.

4. HISTORY

Floods along the Aroostook River have occurred to varying degrees over the years resulting from intense rainfall, snowmelt and ice jams or from combinations of the three. The main flood season on the Aroostook River occurs in the spring when heavy rain accompanied by snowmelt combined to cause considerable runoff; fall season floods which can occur from rains accompanying coastal hurricanes and tropical storms are generally lower in magnitude than spring floods. Ice jams in the Aroostook River cause a major flood hazards most every spring. Most notable past historic floods are the April 1973 and April 1983 events.

The recent April 3, 1986 ice-out flood stage caused road inundation and flooding of commercial property along Main Street with as much as 3 feet of water at one property.

According to local officials and historical records, a recurring April 1973 event in the project study area would cause an estimated $500,000 in average annual damages to commercial and residential property, the adjacent railroad, and to downtown streets in Fort Fairfield. The April 1973 event had an estimated discharge of 58,200 cfs at Fort Fairfield.

5. PROJECT DESCRIPTION

The selected plan for local flood protection in Fort Fairfield consists of an earth dike approximately 2,900 feet long with stoplog railroad gates at each end, a 65 cfs capacity pumping station for low level interior drainage behind the dike, and a 4-foot diameter pressure conduit for high level drainage. The dike extends from just downstream of Limestone Road bridge at top elevation 370.5 feet NGVD and continues easterly along the riverbank, extending to high ground at a point just upstream of the railroad station in downtown Fort Fairfield at top elevation 369.0 feet NGVD. The dike should provide flood protection to commercial and residential properties on the north side of Main Street. (See Plate 7)

The top of dike will vary approximately 15-20 feet above existing ground with a top width of 12 feet. The dike core will be compacted impervious fill. The riverside and landside slopes will be 1 vertical to 2.5 horizontal. The riverside slope will have a dumped gravel toe berm. Stone protection (1.5 feet thick) will be placed on the toe and a 1-foot gravel bedding layer underlain by the compacted impervious fill above the toe berm. Stone sizes will be approximately 1-foot in diameter except at the transition sections where it will be approximately 2 feet in diameter. The landslide slope will be protected by 6 inches seeded topsoil and a gravel toe trench.

6. TAX LOSS

The anticipated tax loss for the Fort Fairfield Local Protection Project, based upon the 1986 tax assessments of the town is estimated to be approximately $1,900.00 dollars, which was furnished by local town officials.

7. ACQUISITION COSTS

Acquisition costs will include costs mapping and surveys, legal description, title evidence, appraisals, negotiations, and closing and administrative costs for possible condemnations. The acquisition costs are based upon this office's experience in similar civil works projects in this general area and are estimated at $3,000 per ownership.

19 OWNERSHIPS x $3,000.00 = $57,000.00

8. <u>RELOCATION COSTS</u>

Public Law 91-646, Uniform Relocations Assistance Act of 1970, provided for equitable treatment of persons displaced from their homes, businesses, or farms by a Federally Assisted Program. In accordance with this law, a sum of $200 per ownership is estimated to cover possible reimbursable expenses incidental to transfer of real estate interests which may be incurred by the ownerships in this acquisition program.

Included among the items under Pl 91-646 are the following:

a. Moving Expenses

b. Relocation allowance (Business)

c. Replacement Housing (Tenants)

d. Relocation Advisory Services

e. Recording Fees

f. Transfer Taxes

g. Mortgage Prepayment Costs

h. Real Estate Tax Refunds (Pro-Rata)

Preliminary surveys indicate that no relocation of existing residential and commercial properties will be required for the proposal project.

ESTIMATE OF THE RELOCATION COSTS

19 OWNERSHIPS X $200.00 EACH = $3,800.00

9. <u>SEVERANCE DAMAGES</u>

Severance damages usually occur when partial takings are acquired which restrict the remaining portion from full economic development. The severance damages are measured and estimated on the basis of a "Before" and "After" appraisal method and will reflect actual value loss incurred to the ownerships as a result of partial acquisition.

Preliminary investigation indicate that no ownership will incur severance damage because of the taking. The acquisition will be under Permanent Easements.

ESTIMATE OF SEVERANCE DAMAGES = -0-

10. <u>PROTECTION AND ENHANCEMENT OF CULTURAL ENVIRONMENT</u>

In accordance with instructions set forth in Teletype DA (DAEN) R 111306A, dated October 1971, Subject: "EO11593, 13 May 1971, Protection and Enhancement of Cultural Environment; and DA AR200-1 dated 15 July 1982; "our preliminary field investigations revealed that no local, State, Federally owned nor Federally controlled property of historical significance would fall within the provisions of EO11593 and AR200-1.

11. <u>ZONING</u>

The lands affected by the project are zoned commercial.

12. <u>HIGHEST AND BEST USE</u>

The highest and best use of the affected lands is considered to be the present use.

13. <u>MINERAL DEPOSITS</u>

A recent field inspection discloses no evidence of commercial mining or gravel nor the deposits of any minerals within the project area.

14. <u>CROPS</u>

Several trees have been killed off either by flood damage or disease. However, the quality and quantity of the healthy growth are considered inadequate to require inclusion of any special allowance for merchantable timber.

Agriculture - There is no evidence of any commercial agricultural efforts in the project area.

15. <u>UTILITIES AND SERVICES</u>

Electric power, telephone, Town water, and sanitary sewers are available to all properties within the project area.

16. <u>WATER RIGHTS</u>

Suggested interim guide lines for shore land zoning and subdivision control have been distributed to municipalities in Maine, and Department of Environmental Protection, State Planning Office. The guide lines are intended to assist communities with municipal shore land zoning.

All buildings and structures except those requiring direct access to the water as an operational necessity shall be set back at least 100 feet from the mean annual high water line.

Those standards may be waived by a municipality because of existing structures, and those requiring direct access to the water as an operational necessity. A recent inspection and discussion with the Town Manager revealed no ownerships in the project area require access to the River for their operational needs.

17. BORROW AREA

No land has been included in this report for borrow purposes.

18. RELOCATIONS - Roads and Public Utilities

No roads but public utilities (sewage) will require relocation. The main sanitary sewer which services this area of Fort Fairfield will be relocated in proposed Permanent Easement Area of this project.

19. CONTINGENCIES

A contingency allowance of 25 percent is considered to be reasonably adequate to provide for possible appreciation of property values from the time of this estimate to acquisition date, for possible minor property line adjustment or for additional hidden ownerships which may be developed by refinement of taking lines, for adverse condemnation awards and to allow for practical and realistic negotiations.

20. GOVERNMENT-OWNED FACILITIES

Section III of the Act of Congress approved 8 July 1958 (PL85-500) authorized the protection, realteration, reconstruction, relocation or replacement of Government-owned facilities. A preliminary inspection of the property area indicated no Government-owned facilities are affected.

21. RIGHTS TO BE ACQUIRED

Local interests are required to provide all lands, easements and rights-of-way necessary for project construction. Appraisals for acquisition will be received by this office.

22. FEE REQUIREMENTS

Preliminary investigations indicate that both improved and unimproved properties will be affected by the proposed Fort Fairfield Local Flood Control Project. Based on Project Engineering Plans, one fee acquisition will be required of the project. Lot 29 consisting of .37± acres of land (16,117 SF) owned by Pineland Development Corporation will be acquired in fee.

Therefore, the fee acquisitions that are necessary for the subject project are estimated as follows:

FEE ACQUISITIONS

LAND .37± acres (16,117 SF x $ .60 PSF) = $ 9,670.20
Call $ 9,700.00

23. EASEMENT AREAS

A. Permanent Easement Areas

Permanent easements for construction and maintenance purposes are necessary. The easement areas adjacent to the waterway vary in width throughout the project area and contain approximately 4.06± acres.

Preliminary investigations indicate that after the imposition of the permanent easement interests adjacent to the waterway, their highest and best use of the remainder of the properties will not be materially affected. However, lands would remain in their private ownerships to maintain conformity with their existing lot requirements.

The following costs for the permanent easement interests are considered fair and reasonable for imposition of the 4.06± acre easement areas.

4.06± acres @ $29,000 per acre = $116,870.00

B. Temporary Easement Areas

Construction measures would require temporary easements for contractor work areas along the entire length of the dike. The required work areas will be about 35 feet wide and will run contiguous to the inboard toe of the proposed dike length of 2,900 feet. Exceptions to their contiguity are at certain points where their close proximity to existing structure. In these cases, the structures will not be affected. The easements would affect about 16 private ownerships, and tow municipally owned parcels.

It is estimated that about 2.33 acres will be required for right-of-way and temporary construction easements. Right-of-ways to the proposed dike and pumping station will be situated on town-owned land which are included in the proposed project. The cost for temporary construction easements is estimated to be about 10 percent (10%) of the estimated market value of the land per year. This amount is predicated on an amount equal to the estimated fair return an investor would be entitled to on invested capital and provision for economic tax. For purposes of this report, it is estimated that the temporary construction easements will be required for one year.

2.33± acres @ $29,000 per acre	$67,570.00
Fair rate of return at 10% per year (for one-year)	x 10%
	$ 6,757.00

24. EXISTING FLOWAGE EASEMENTS

The Maine and New Brunswick Electrical Power Company Limited constructed a dam about 1908, known as Tinker Dam, downstream from Fort Fairfield on the Aroostook River in New Brunswick, Canada.

Since the dam was constructed, there have been 70 or more flood damage claims filled with the power company alleging the damage was due to the fact that the dam caused the flooding. According to records of the power company, when these claims were settled they attempted to secure flowage easements over these properties. At least in some cases they were able to secure a flowage easement which reads in part:

> "The right in perpetuity to flow from time to time as the needs of the Grantee, its successors and assigns may require to such heights as they may be flowed by the maintenance of the Grantee's existing dam at Tinker in the said Province of New Brunswick at its present elevation with flashboards at the level of 498 as established by a brass plug in the cutoff wall of the head works of said dam, the Grantors' premises situated in said Fort Fairfield bounded and described as..."

Pending a detailed title examination of each ownership involved, it would be difficult to identify which ownerships have flowage easements to the extent thereof.

25. EVALUATION AND CONCLUSION

A thorough search of the records was made in the Town of Fort Fairfield, Maine to obtain comparable sales data. In addition real estate brokers, local officials, and knowledgeable persons were interviewed to obtain data and value estimates. This evaluation is based upon the knowledge of the general real estate market in the area which was obtained from this study and analysis. All of the properties affected within the project area have been inspected from the exterior. A random sample of interiors were also inspected when owners were interviewed.

The trend of property values in the Town of Fort Fairfield appear to be static as evidenced by the few new construction starts and limited real estate transfers. For the most part, business properties and commercial establishments that are affected by the proposed project purchase area have remained in the same family ownerships for many years.

The assigned values used in this estimate are for the most part considered nominal which reflect both small and large tracts of land with differing characteristics. Based on this fact real estate market values are estimated at $29,000.00 per acre, with a square foot market value of $.60 PSF, due to its characteristics.

26. <u>GROSS APPRAISAL</u>

The following is a summary of the real estate required; its estimated market value:

<u>FEE</u>

<u>IMPROVEMENTS</u>

None -0-

<u>LAND</u>

0.37± Acre Commercial Land
(16,117 SF @ $ .60 PSF) $ 9,670.20

Total 0.37± Acre Cost Land & Improvements (Fee) 9,670.20

Call $ 9,700.00

<u>PERMANENT EASEMENT</u>

4.06± Acres Commercial Land
@ $29,000 per acre $116,870.00

Total 4.06± Acres Cost Land (Permanent Easement) 116,870.00

<u>TEMPORARY EASEMENT</u>

2.33± acres @ $29,000 per acre $ 67,570.00
Fair rate of return at 10% per year (for one-year) x 10%

Total 2.33± Acres Cost Land (Temporary Easement) $ 6,757.00

COST SUMMARY

The following is a summary of the total estimated real estate costs of the proposed project:

Total Cost 13.18 Acres Land & Improvements (Fee, Permanent & Temporary Easements)	$133,327.00
Severance Damages	-0-
Relocation Assistance	$ 3,800.00
Acquisition Costs	$ 57,000.00
Contingency Allowance (25%)	$ 33,163.00
TOTAL ESTIMATED REAL ESTATE COSTS	$227,290.00
ROUNDED TO	$227,000.00

COMPARABLE SALES - FORT FAIRFIELD & AROOSTOOK COUNTY, MAINE

SALES NUMBER	DATE OF SALE	LOCATION	GRANTOR	GRANTEE	LAND AREA	ZONING	PRICE
1	Dec. 1986	Ft. Fairfield, ME	W. Adams	K. Thibeau	1.43±	Comm.	$29,800.00
2	Aug. 1985	Caribou, ME	M. Carter	R. Deschene	1.0±	Comm.	$37,000.00
3	Aug. 1986	Ft. Fairfield, ME	Dupree Realty Trust	D. Wilcox	1.0±	Comm.	$29,000.00
4	Apr. 1986	Presque Isle, ME	L. Roberts	C. Walton	1.06±	Comm.	$25,000.00